Engineering Record

A Weekly Journal Devoted to Civil Engineering and Contracting

PUBLISHED BY MCGRAW PUBLISHING COMPANY, INC.

Volume 69 NEW YORK, SATURDAY, APRIL 18, 1914 Number 16

Error as to Crystallization

IT was stated editorially in this journal on Dec. 13, 1913, that a 330-ft. bridge span, removed after being in service since 1885, had shown evidence of crystallization in the tension members. For that reason, the note continued, the old eye-bars were replaced when the bridge was re-erected. The statement as to crystallization was immediately questioned by one American and one German steel specialist. Efforts were made, therefore, to get full data, whereas previously only statements, that seemed reliable, were available. Only now have the exact facts been ascertained, and it can be stated with certainty that they do not form any basis for the belief that there was crystallization in service. While the fractures of eye-bars, tested to destruction, were granular, this was undoubtedly due to treatment in manufacture.

"Civil Engineer" Defined

WHILE engineers in the United States have been spending their time in argument as to the advisability of licensing laws their brethren in British Columbia have been acting. As noted elsewhere in this issue they have succeeded in putting into the water-law of the Province a definition of "civil engineer" which accords with the suggestion of many engineers on this side of the line—that the necessary qualification be membership in a national engineering society. Much has been heard in favor of vesting, through licensing laws, the control of the profession in the hands of its members. Certainly this desideratum is accomplished, so far as it goes, in the British Columbia definition. It states that a "civil engineer" shall be a member of the Canadian Society of Civil Engineers, of the institutions of civil engineers of Great Britain or Ireland or of a civil engineering society of equal rank in the British Empire or any foreign country. While such a definition would be objected to by those not members of a large society, it will have the hearty support of the majority. The engineers of British Columbia are to be congratulated on their success.

New York's Water-Power Situation

HYDROELECTRIC legislation in New York State, much agitated this year, though not with as much vigor as in 1913, will not materialize in this Legislature. The regular session adjourned without taking action on the bills before it and now in calling an extra session the Governor names only the State's finances for the consideration of the members. Those in favor of sound financing of water-power development will rejoice that action is delayed another year, but they could take more comfort were there evidences that sane thinking on this subject is on the increase. On the contrary, the politician-advocates of State development have continued their ranting against existing central-station companies, and their citation of the inconclusive figures of the Provincially operated system in Ontario and the absurdly low estimates for State development. Possibly relief lies on the political horizon, for, unless re-elected, the present Legislature (excepting half of the senators) and the Governor go out of office this year. The Governor is avowedly in favor of State development, though the complexion of the Legislature made passage of the legislation doubtful.

Quarterly Waterworks Journal

IN abolishing its yearly volume of proceedings and substituting therefor a quarterly journal, the first issue of which has just been distributed, the American Waterworks Association has rendered a valuable service to its members. While not by any means ideal as a news medium, it will be immensely superior from the news point of view to the present annual volume, which necessarily is only a repository of what has transpired. In outlining the plans for the new journal, John W. Alvord, chairman of the publication committee, has stated that it is the purpose to have the first number appear early in the year, prior to the annual convention; it will contain papers which will later be discussed at the meeting. The second number will contain the proceedings of the convention. The third and fourth numbers will complete the discussions, and contain additional papers submitted to the local sections and announcements of local meetings. It is hoped by this procedure to present more promptly any papers of merit and furnish a repository for waterworks literature. The typographical appearance of the March number is excellent and the publication committee is to be congratulated upon the results of its first efforts.

Wreck Due to Ignorance

THE wreck of the Wabash Bridge at Attica, Ind., described in the Current News Section of this journal last week, was, so far as can be learned, utterly inexcusable and can be explained only on the ground of most lamentable ignorance. It has not even the "did-not-know-it-was-loaded" excuse. Shortly after a vital injury to the structure, a passenger train was allowed to enter the bridge. Had this occurred in an inaccessible locality, in storm or in darkness or under great stress of circumstances, it might have appeared less flagrant, but the disaster took place in broad daylight, where communication was open and after a deliberate investigation of conditions. The cover plate of the end post was split for a considerable distance, thus entirely destroying its integrity as a strut, and changing the compression section to a pair of limber webs, capable of developing only a fraction of the compressive strength of the original member. Any sensible man would have forbidden the use of the structure. Nevertheless, a heavy live load was applied and because the ignorant observer failed to note any deflection the bridge was pronounced safe. Under the circumstances, deflection or its absence was no criterion of the condition of the bridge. Its failure was not, and could not be, due to deflection but to the simple failure of the crippled member and consequent destruction of truss action. Just how far the higher authorities are to blame for the accident is difficult to say. That depends on the instructions given the supervisors and the system by which their full understanding of their duties is assured. Certainly all will grant that the supervisor should have known that the structure was in an unsafe condition. Why he did not—and on whom rests the blame—will likely be brought out in the court proceedings that will probably follow.

Why the Railroad Evaluation?

DECIDED objection was made by a number of members of the American Society of Civil Engineers in the recent valuation discussion to the committee's assumption that the Federal evaluation is for the purpose of rate-making. Strangely enough this objection was most strongly made by the valuation engineers of the railroads, who seemed to think that such an attitude was unwarranted, indiscreet and even dangerous. The criticism does not appear to be based on a careful reading of the valuation act, which says that "all final valuations by the commission and the classification thereof shall be published and shall be prima facie evidence of the value of the property in all proceedings under the act to regulate commerce." If further elucidation is required it is to be had in the recent address of Charles A. Prouty, director of valuation, before the Chamber of Commerce of the United States. "Under the constitution as interpreted by the United States Supreme Court," he said, "the property of public utilities must be allowed a fair return upon the fair value of the investment. . . . Manifestly it is not a difficult thing to determine the rate of return to which this property is entitled. There still remains, however, the value upon which this rate is to be computed, and until that value is known it is impossible to determine what total income the property is entitled to earn and, therefore, fix the charge which may be justly made against the public." While no one will contend that the valuation will be the only factor in fixing the rate, certainly, in the face of the above quotations, there seems to be no ground for conjecture as to the relation of the valuation to rate-making.

Getting Action on Reports

FAILURE of municipal authorities to follow expert advice in strictly engineering matters is not tolerated in Europe, but many American officials feel that they are competent to pass on a city's distribution system, albeit they would hesitate to enter the surgeon's domain and operate for appendicitis. There are, of course, cases where the money cannot be obtained, or only sufficient funds for a smaller project than the one recommended. Then again there are political debts to pay, but when the costs of alternatives are about the same and the ends to be attained are strongly in favor of the scheme recommended, contrariness or ulterior motives seem the only reasons for choosing the less favorable proposal. Publicity ought to help immensely in such cases.

Elsewhere in this issue is described a simple plan by which Cincinnati can avoid the repetition of the water shortage it experienced last December, during the four days following a break in its only supply to the business section. A proclamation of the mayor closed down factories and restricted the use of water to domestic consumption. Fortunately, no fires occurred. As a result there was a demand for better protection, and the suggestion that a 36-in. main be laid back of the city had many adherents. An engineer was employed to study the situation and propose a remedy. He has reported that the proposed 36-in. line would be nearly 13 miles long, must be laid through many other municipalities and would not give an amount of water commensurate with the cost. He shows that the laying of 3 miles of 60-in. main at a considerably lower cost will answer every requirement. The choice between 13 miles of 36-in. and 3 miles of 60-in. conduit is easily made, especially when future ordinary demands will necessitate the construction of the larger main within a few years. Local engineering societies, where they exist, can help much in securing the adoption of reports so clear-cut as this one. Were there such an organization in Cincinnati, it would now be in a position to render a great service to the community.

Ottawa's Water-Supply Controversy Ended

BUFFETED about upon a turbulent sea of politics for four years, the new water supply plan for Ottawa has apparently reached port. As announced in the Engineering Record recently, the citizens, by popular vote, have selected the project for purifying Ottawa River water by rapid sand filters, thus bringing to an end one of the bitterest controversies in the history of municipal waterworks. The entire proceedings at Ottawa suggest nothing so vividly as the vagaries of the course of a ship steered by a drunken helmsman and manned by a crew of mutineers. The city officials have been traveling in circles and are now back at a point where they were in 1910 when Allen Hazen proposed rapid sand filters for Ottawa River water, as a substitute if the McGregor Lake project was defeated. A brief review of the controversy will be found on page 440.

One of the interesting points brought out by the controversy is the effect which Government officials and scientists in lines unrelated to waterworks may have on the voters. In Ottawa there are many of these Government officials and letters from them were used extensively in the papers. Opinions, not justified by the facts, were freely advanced, the effect of which, owing to the standing of the writers, was great.

There seems to be no doubt that a splendid supply could have been obtained from the Thirty-One-Mile Lake region, but from the reports of Mr. Hazen and a majority of the commission it seems clear that a supply superior to almost any in the United States can be obtained from McGregor Lake at a greatly reduced cost. It further appears that a very satisfactory supply, better than that of a large majority of cities in the United States, can be obtained from the Ottawa River by rapid sand filtration. Inasmuch as the cost of the Thirty-One-Mile Lake project is so greatly in excess of either of the other two schemes, it is certainly an open question whether the city would have been justified in spending an unnecessarily large sum for such a supply. As between the Ottawa River and the McGregor Lake schemes, there seems to be little difference in cost. The McGregor Lake system in first cost would be about double that for rapid sand filtration for the Ottawa River. The operating cost, however, including filtration and pumping, would make the net yearly cost to the citizens of Ottawa not materially different, with perhaps the advantage at present in favor of filtration, and in future, as the supply becomes greater, in favor of McGregor Lake.

After squandering thousands of dollars on reports and investigations Ottawa has, for the time being at least, decided what it wants. The city in the beginning had competent advice. It has dearly paid the penalty in lost lives and money, in not following it.

To Obviate the Opening Up of Pavements—A Suggestion

PAVEMENT piercing for petty repairs by public utilities is so universally practised in our larger cities that the public has become inured to the spectacle of seeing the paving materials on the sidewalk or piled in the gutter. As this is written there are two holes in front of this building—the Old Colony in Chicago—and yesterday one was closed on the side street. True, the excavation is carried on inside a rail and the repaired pavement is about as good (or bad, and insanitary in this case) as it was before the workmen started prying it up. At least once a month the same thing has occurred in the same pavement for years, and hardly a block of new pavement in downtown Chicago has the top surface scratched by traffic before it is cut into.

It has been said that the contractors rarely figure on making much if any profit on the original laying, because the remuneration for the subsequent patch and repair work done at a fancy figure before the end of the guarantee period more than makes up any original loss. Fortunate is the paving contractor who finds that a skyscraper is to be erected adjacent to one of his streets, for it is more than likely that the building contractor will be required to repave the whole street.

Inquiry fails to disclose any attempt to get away from the once-down-for-all-time type of pavement, and the Engineering Record suggests that engineers look into the feasibility of a removable type of pavement which can be picked up in large blocks extending from the curb to the car track and laid to one side while the delver after conduit troubles pursues his task.

Inquiries in Chicago and talks with practical pavement men indicate that the idea has merit and presents few if any insurmountable difficulties if applied to any downtown street having car tracks from which a mounted crane might manipulate the blocks. Rough calculations indicate that concrete blocks of 10-in. thickness, reinforced near both faces so that both sides might be used, armored on the edges, made of very rich mixtures, with granite aggregates, in a slab plant, practically under laboratory conditions, might cost upward of $2.70 per square yard, exclusive of hauling to place. They could be transported at night over the street-railway tracks and unloaded by a crane directly onto previously prepared longitudinal reinforced foundation walls at curb and car tracks.

Blocks would seem to be specially suited for the portion of the street that the railway companies are required to pave, as in most cases foundations for the blocks already exist. The almost inevitable necessity of removing the pavement prior to its life limit should make the removable, reversible and interchangeable features worthy of detailed study by street-railway engineers.

As for handling the problem of subsequent small street excavations complained of in the opening paragraph, a simple hand-operated gantry crane should be able to raise one block to the top of the next. Should long openings be desired, the street-car crane would be requisitioned. To any one who has witnessed the placing of 30 and 40-ton blocks in the unit subways of Kansas City, or in the Burlington railroad bridges, the task of setting a 5-ton block, 6 ft. wide by 14 ft. long, on a city street seems mere child's play.

Discussion of the suggestion is invited.

Important Features of the New Quebec Bridge

THE design of the new Quebec Bridge, as was expected, embodies not only a number of important features developed in the largest spans previously built but also some new ones of a radical character. This statement applies to both the main 1800-ft. span and to the 515-ft. anchor arms. Some of the important details and departures are described in an article in this issue.

In the general truss design, the K-shape bracing has important advantages in distributing the shear between two members in each panel, thereby reducing the sizes of the members, simplifying the connections and facilitating field assembling. A decided advantage in erection, too, is found in the very interesting departure of providing separate pins for the connections of different members at main panel points, a feature, which, of course, also improves the fabricating details. Shopwork is further facilitated by the use of straight instead of curved chords, with the necessary consequence of right-angle joints in the compression members.

In the center span and cantilever arms, where weight reduction was important, the selection of nickel-steel for many members was to be expected. In the anchor arm the connection of the vertical anchor bars to the top chords increased the length of the latter, providing better for longitudinal temperature movement, and possibly reducing secondary stresses.

The eye-bar arrangement in the chords is especially worthy of note. The bars, some of which are 85 ft. long, are supported intermediately by special lattice girders. The excessive stresses in the pins, had the usual method been followed, and the difficulty of packing and of assembling the numerous bars in each panel are overcome by placing them in two tiers with double pin connections to the vertical posts.

On account of the part played by the lower-chord design in the previous structure, the section adopted for the new bridge is of special interest. It consists of four deep webs, latticed top and bottom, and having horizontal plate diaphragms on the longitudinal center line.

The secondary stresses and difficult connections, due to the riveted joints at the ends of the enormous single and double-web floorbeams, have been avoided by pin-connecting them to the vertical members. Symmetrical loading is obtained by placing the double webs on either side of the webs of the vertical posts and by placing the single webs between the double webs of the hangers. All pins subject to turning have full-length bushings. All transverse forces have been concentrated in the planes of the lower chords by the elimination of the top chord lateral system and by the combination of vertical and inclined sway-braced system transmitting the horizontal forces directly to the bottom-chord panel points.

These and other special features indicate the substantial progress that has been made in this design and suggest the direction in which future advances will be made.

The Latest Street Railway Car

THE appearance of the stepless center-entrance car in a number of American cities is an event of more than casual importance to the engineer. For many years cars have been designed along empirical lines, but now the structural engineer has become a factor of no mean importance. When the ordinary end-platform car was made of wood any unlettered carpenter felt competent to build one. By and by somebody discovered that a little steel plating for the main-floor framing would add greatly to the strength of the structure. Next, about the year 1903, the all-steel underframe appeared, but even then judgment as to the size and weight of the members embodied more guesswork than science. Perhaps the all-steel cars built for the New York subway were the first which showed evidence of real car engineering. But it remained for the coming of the stepless center-entrance type to place car design on as true an engineering basis as bridge building.

The essential feature which makes the center-entrance car possible is that the stresses which had hitherto been carried by deep floor members, side posts and roof ribs (carlines) are now taken by side-girders, which are stiffened by the posts and a few angles. These girders are cut at the center for the door opening, but enough metal is left above and below the doors to care for the stresses. The transfer of the load to the side girder in this fashion makes it practicable to build the floor of the car in several planes, thus abolishing the high step which accompanied the use of 33-in. wheels and large motors.

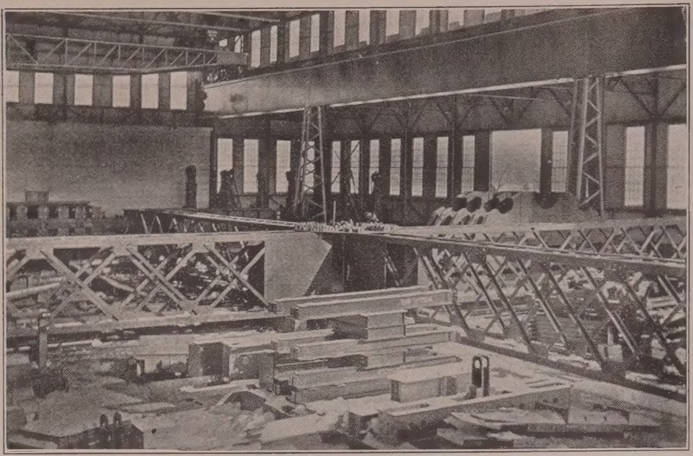
The side-girder design has done more than to make the low step possible, for it has made possible the reduction of the dead weight per seated passenger from 1000 or 1200 lb. to less than 700 lb. Electric railways now realize that a car designed by a structural-steel engineer and not by a rule-of-thumb mechanic means real economy both in first cost and in energy consumption. While on the question of structural material, it may be worth noting that wood, whether carved or plain, has been banished for all car purposes except seat framing.

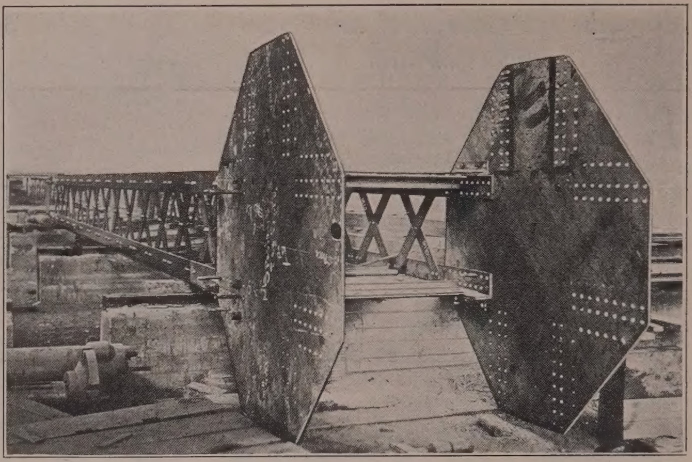
This progress toward scientific design and sanitary fireproof material has been coupled with wonderful betterments in the operating equipment and furnishings. The awkward form of direct-platform control has been replaced by an indirect system in which the main currents are noiselessly and safely broken in switchboxes under the car, the motorman manipulating only a very small low-voltage master controller which sets the main circuits in action. This controller is furnished with a spring button (deadman's handle) which will cause the opening of the motor circuit and the application of the airbrakes at emergency position in case the motorman relaxes his pressure on the button while the controller is in any running position. This safeguard, which was devised originally for train operation, is reinforced by the electrical interconnection of the passenger doors and the controller in such a way that the car cannot be started until the doors are closed; nor can the doors be opened before the car has come to a full stop. Airbrakes are, of course, used instead of handbrakes, though the latter are provided for emergency use. On the whole, the motorman can now do much better work not only because of the improved control and braking apparatus but also because he is now enabled to do his work in a cozy cab to which passengers have no access at any time.

The conductor is also benefited by the modern center-entrance, pre-payment car. He is not exposed to trying weather, is not obliged to fight his way through the car for fares and is stationed at a point where he can control the entrance and exit of passengers through air-operated doors. Fares may be registered by the mechanism of the fare box or the conductor can record them electrically on an ordinary register by means of push-buttons at his stand. As the gong or light signals which the conductor gives to the motorman are also operated through push-buttons, the interior of the car is free from the usual register and signal cords, and the conductor is relieved from the fatigue of raising his arms to these cords several hundred times a day. The conductor stands opposite air-operated doors, which are glazed to permit him to see conditions in the street. It is impossible to enter or leave the car while it is in motion, and push-buttons at each seat allow the passenger to signal that he wants to leave the car at the next crossing.

Last but not least, the heating, ventilation and lighting of the modern car represent great steps forward. So long as the old two-level monitor roof was used, satisfactory ventilation in cold or rainy weather was impossible. If the deck sashes were open, drafts, rain and snow annoyed the passengers; and if they remained closed, the air soon became noisome and stuffy. The adoption of the arch roof made a change to mechanical ventilation mandatory. In the modern ventilating systems the motion of the car or the operation of an intake fan first draws air through openings in the sides and then passes it over the electric heaters, the exhaust going out through registers in the roof. It will be seen that this plan combines both ventilation and heating. Lighting, too, is being changed from the clear-glass carbon incandescent system to the more pleasing semi-indirect or indirect forms of lighting with the economical tungsten lamp. If any more proof were needed to show what a good return the American citizen gets for his five-cent fare it might be added that the center-entrance car uses the area otherwise required for platforms, not for more standing room but for more seats.

It should be emphasized that the foregoing remarks refer to a composite of the present center-entrance cars rather than to any one of them. Moreover, some of the improvements, not even excepting the stepless feature, have been applied to end-platform cars. Nevertheless, the stepless center-entrance car deserves to be considered as the one which is most typical of modern street railroading.


Bottom Laterals Assembled in Bridge Shop


Bottom Lateral Strut and Intersection Plates

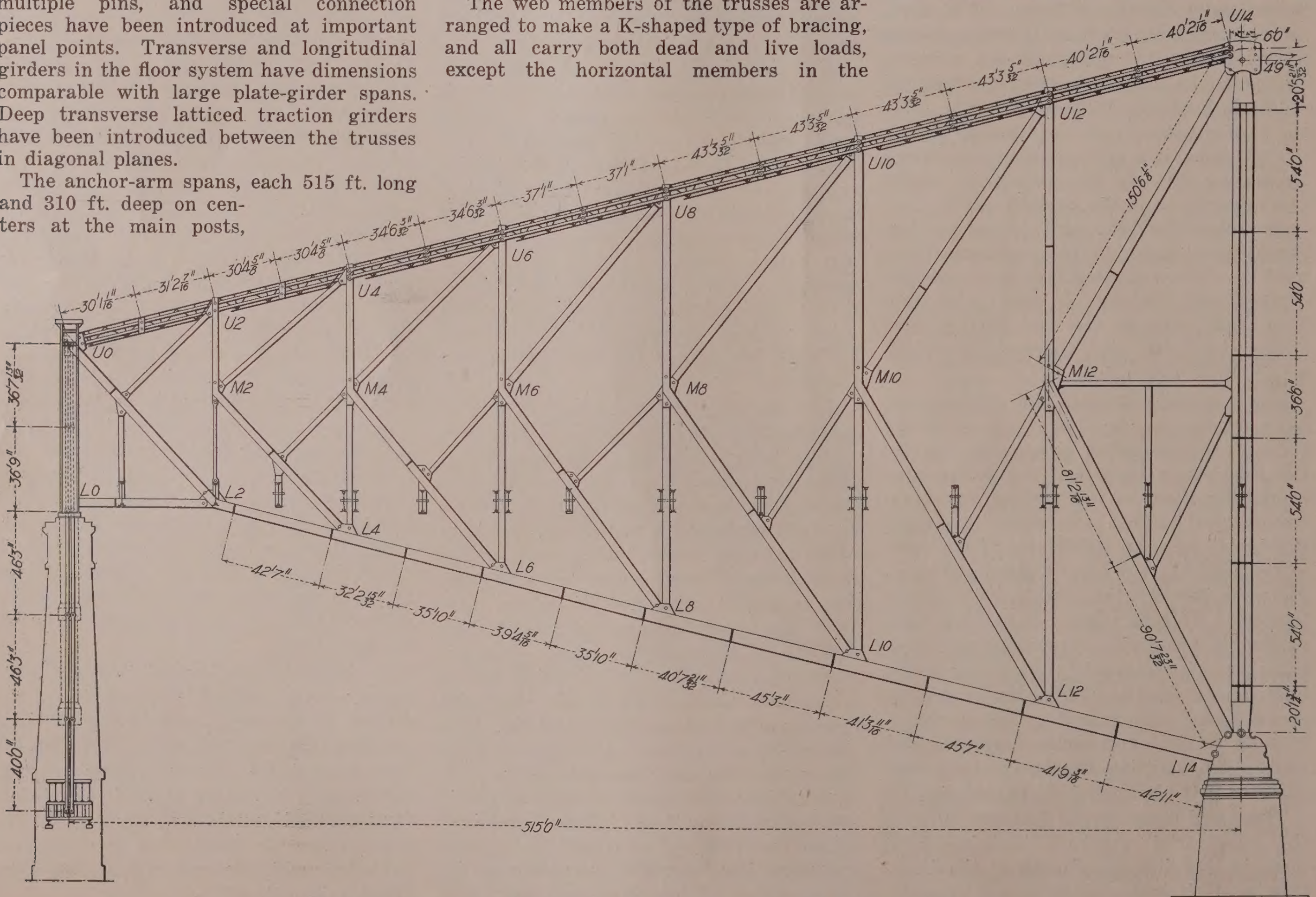
Quebec Bridge Anchor-Arm Spans

Important Developments in Main Trusses, Bracing, Twin Tension and Compression Members, Multiple-Pin Connections and Field-Spliced Shipping Units

IMPORTANT modifications in both type and details of long-span steel-bridge construction have been developed in the 2830-ft. double-track Quebec Bridge. Many of them appear in the anchor arms and are described in this article. The type of main-truss bracing is special. The usual top lateral system is eliminated. The extraordinary dimensions and great weight of the truss members necessitate shipping some of them in sections, afterward united by field splices to make the panel units. Auxiliary trusses have been provided to support long flexible members, main panel-point connections have been made with multiple pins, and special connection pieces have been introduced at important panel points. Transverse and longitudinal girders in the floor system have dimensions comparable with large plate-girder spans. Deep transverse latticed traction girders have been introduced between the trusses in diagonal planes.

The anchor-arm spans, each 515 ft. long and 310 ft. deep on centers at the main posts, have two pin-connected trusses 88 ft. apart, and weigh 16,600 tons each, exclusive of the steel in the anchor piers, but inclusive of the 2700-ton main vertical posts and their 900-ton bearings, which are common members in them and in the cantilever arms. The trusses are each proportioned for a live load of two E-60 locomotives followed, preceded or followed and preceded by a train load of 5000 lb. per linear foot. They are made entirely of carbon steel, with the exception of the end diagonals L_2U_0 and the connection details at L_2 and at U_{14}, which are made of nickel steel.

The web members of the trusses are arranged to make a K-shaped type of bracing, and all carry both dead and live loads, except the horizontal members in the center of the panels adjacent to the main posts on the cantilever side, and the trusses supporting the top-chord eyebars. The chords are in straight lines. None of the principal stresses are indeterminate. The long double-top-chord eyebars are supported at the center of each panel on special lattice girders connected to the tops of the vertical posts. The 7 x 10-ft. bottom chords are made of four built I-beam ribs laced together, and with horizontal bracing on the center line, the webs being held in place by transverse diaphragms. The principal compression members have intermediate field-riveted transverse splices, and in some of them the sections between these splices are made of two pieces shipped separately. All web members are made up of plates and angles and are pin-connected at their intersections. At the main panel points the web members are connected by



Erection Diagram of Anchor-Arm Truss, Showing Field Splices of Members pins through gusset plates riveted to the bottom chords, independent connections being thus provided for each member.

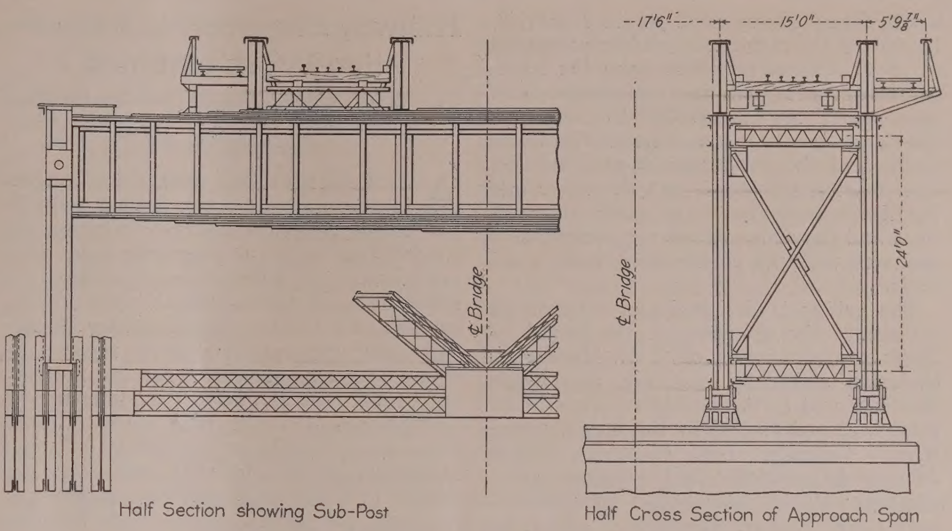
Some of these features are of special interest in comparison with the 500-ft. anchor-arm spans of the original Quebec Bridge. The trusses of the latter were 315 ft. high and 57 ft. apart and weighed 6250 tons for each span, exclusive of the 700-ton floor system. Both top and bottom chords were parabolic curves, and the trusses were divided into 100-ft. main panels and 50-ft. sub-panels. The types of truss, secondary bracing systems, top-chord and web-member design and floor system conformed in general to standards for ordinarily long spans. The bottom chords were made of four built channels having their flanges connected by tieplates and lattice angles. They were about 4½ ft. deep and 5½ ft. wide, with a maximum cross-sectional area of 776 sq. in., as compared with a maximum area of 1902 sq. in. in the present structure. The heaviest chord piece, 65½ ft. long, weighed 165,000 lb. All of the chord joints were oblique to the axes of the members, as described in the Engineering Record of Aug. 10, 1907, page 169.

SECONDARY BRACING AND END ANCHORAGE

THE system of transverse bracing in the anchor and cantilever arms of the present bridge is unusual in that it does not require top laterals. The trusses are connected by transverse sway bracing in the planes of the upper sections of the vertical members and in the lower sections of the inclined members, thus transmitting all transverse stresses directly to the X-braced bottom lateral system in the plane of the lower chords. There are horizontal transverse braces at the top and bottom-chord panel points which, like the diagonals and bottom laterals, have rectangular cross-sections made of four large corner angles latticed together with diagonal angles. All of the bottom lateral diagonals are tapered to correspond with the varying depth of the lower chord.

As all floorbeams are located at a considerable height above the bottom chords, the vertical transverse panels under them are braced by diagonal struts reaching from the ends of the floorbeams to the centers of the bottom horizontal transverse struts. The diagonal braces between the upper parts of the vertical posts are connected at their intersections to short vertical members supporting the horizontal transverse struts below them.

The upper part of the masonry in the shore pier, together with the structural steel inclosed in it, is designed to resist a maximum vertical uplift assumed to be 8,000,000 lb. for each truss under extreme conditions. This is provided for by two sets of vertical eyebars connected to the ends of the top chords at U_0. These eyebars have a combined length of about 206 ft. from their connection with the truss to their connection with the reaction girders. Of this distance 89½ ft. are in an open well in the masonry, which, added to 77 ft. from the top of the masonry to the top of the truss, gives a length of 166½ ft. that is free to move longitudinally in accordance with temperature expansion without disturbing the 40-ft. lower section of bars concreted solidly in position. The lower ends of the anchor eyebars are pin-connected to transverse distributing girders, forming part of the reaction platform, built into the lower part of the pier masonry.



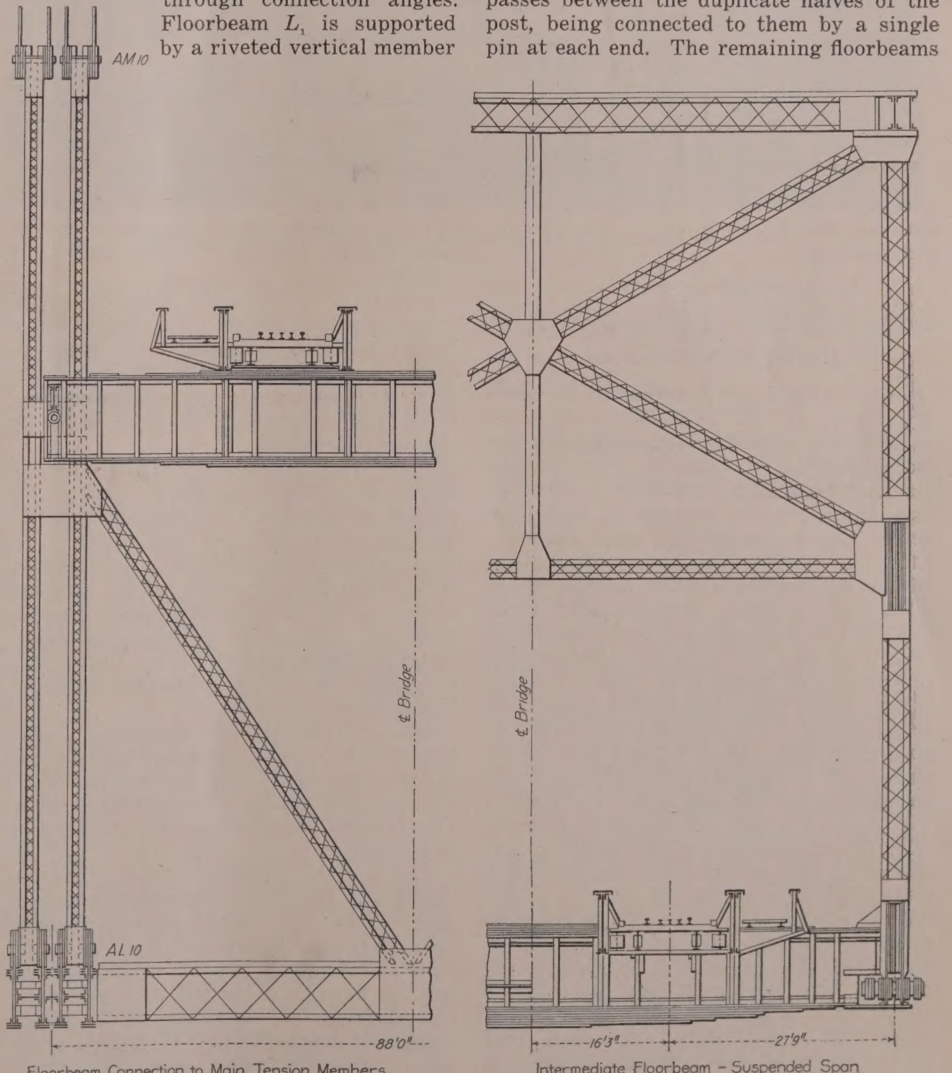
Half Section showing Sub-Post

Half Cross Section of Approach Span

Part Sectional Elevations of Anchor-Arm and End Spans

The bridge has a clear height of 150 ft. above extreme high water. The floor is level on the center span and has a 1-per cent grade on the anchor and cantilever arms. The lower chord is level with the floor at the anchor piers and about 110 ft. below it at the river piers. The floorbeams are therefore connected to the trusses in relatively different positions, which govern their type and the manner of connection. At panel points L_1 and L_2, nearest the anchor pier, the floorbeams are field-riveted through connection angles. Floorbeam L_1 is supported by a riveted vertical member pin-connected to the end diagonal U_0L_2. Floorbeam L_2 is connected to the lower chord. All other floorbeams are connected to the web members of the trusses by 11-in. pins in 15-in. steel bushings.

At the main panel points 4, 6, 8, 10 and 12 the double floorbeams are made of duplicate girders, one on each side of the vertical hangers, to which they are connected by a common pin. At the main vertical post L_{14}-U_{14} the floorbeam is a single-web girder in the center plane of the post and passes between the duplicate halves of the post, being connected to them by a single pin at each end. The remaining floorbeams

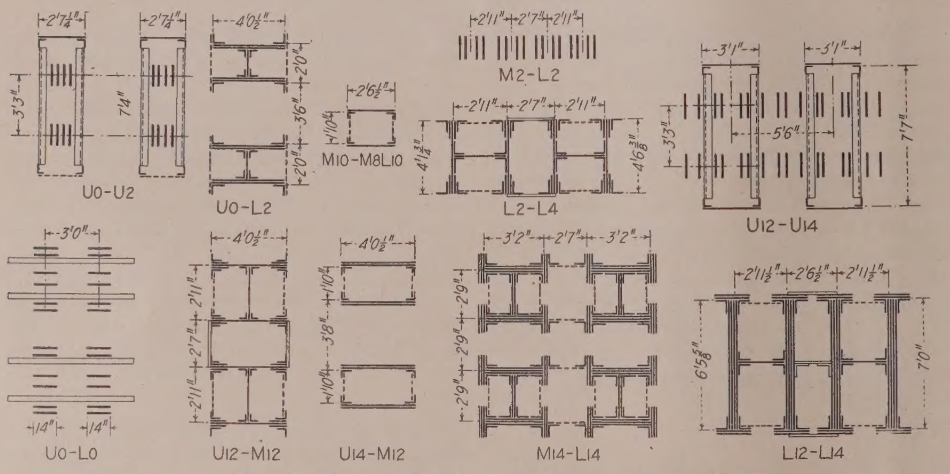


Floorbeam Connection to Main Tension Members

Intermediate Floorbeam – Suspended Span

Half Cross-Sections of Anchor Arm and Center Spans at the intermediate panel points 3, 5, 7, 9, 11 and 13 are single-web girders connected to short vertical members from the intersections of the main and sub-diagonals in each panel. At the shore end of the span the floorbeams are located below the intersections of the sub-diagonals and the vertical tension members. At the river end the floorbeams are above their intersections and the sub-verticals are compression members with pin connections to the diagonals.

The bridge is designed and constructed subject to the approval of the Board of Engineers, consisting of C. N. Monsarrat, chairman and chief engineer, and Ralph Modjeski and C. C. Schneider. It is being fabricated and erected by the St. Lawrence Bridge Company, Ltd., of which Phelps Johnson is president; G. H. Duggan, chief engineer; W. P. Ladd, shop superintendent; G. F. Porter, engineer of construction; S. P. Mitchell, consulting engineer of erection, and W. B. Fortune, erection superintendent.



Typical Cross-Sections of Principal Anchor-Arm Truss Members

Large Lift-Lock Contemplated for Berlin-Stettin Canal

A LIFT-LOCK for a difference in elevation of 118 ft. to accommodate boats of 600 tons capacity is contemplated for the Berlin-Stettin Canal in Prussia. At the site there is in operation a flight of four locks but the increase in traffic requires the duplication of their capacity. After due consideration it was found advisable to construct a lift-lock instead of another flight of locks. It will consist essentially of two horizontal balance beams, each joined by two twin parallel box girders, supporting at their ends tanks for the boats. These tanks will have a capacity of about 60,000 cu. ft. and a total weight of 2200 tons each. One of them may be replaced by a counterweight should it be desirable to use only one tank. The girders will rest on a support that will also receive the articulations of four pairs of levers, which will maintain the chambers in a vertical position during the rotation of the balance girder. Hydraulic brakes are to be provided to control the movements. It is calculated that 75 hp will be sufficient to operate the machinery, but an electric motor of 100 hp will be installed. The power will be applied to rack sectors fixed to the plates of the hydraulic brakes. A second motor of 100 hp will be installed as a reserve.

Railway Engineering Association Work Outlined

Courses of Investigation for the Coming Year Assigned to the Various Committees by the Board of Direction

ASSIGNMENTS of work for the nineteen standing and two special committees of the American Railway Engineering Association for the coming year have been announced. In addition to a general instruction to all committees to make critical examination of the subject matter in the "Manual" and submit definite recommendations for changes, work for the individual committees is outlined as follows:

Roadway.—(1) Continue study of unit pressures allowable on roadbed of different materials, conferring with committee on ballast and special committee on stresses in track; (2) submit specifications for protection of slopes by sodding or otherwise; (3) recommend means for prevention or cure, as case may be, of water pockets in roadbed.

Ballast.—(1) Continue investigation of proper depth of ballast of various kinds to insure uniform distribution of loads on roadway, conferring with other committees; (2) continue study of ballast sections, with particular reference to use of sub and top-ballast.

Ties.—(1) Continue study of effect of design of tieplates and track spikes on durability of crossties; (2) continue study on economy in track labor and material effected through use of treated as compared with untreated crossties; (3) continue study of metal, composite and concrete crossties, building up a history of same; (4) investigate and report on future timber supply for ties; (5) report on distribution and care of crossties.

Rail.—(1) Recommend standard rail sections; (2) continue investigations of rail failures and deduce conclusions therefrom; (3) continue special investigation of rails; (4) present specifications for material in rail joints.

Track.—(1) Present typical plans for double-slip crossings, double crossovers and guard rails; (2) study relation between worn flanges and worn switch points, with a view to correcting the causes and decreasing the number of derailments due to the combination; (5) continue study of economics of track labor; (4) report on design of manganese frogs and crossings.

Buildings.—(1) Report on methods of heating and lighting and sanitary provisions for medium-sized stations; (2) continue study of advantages and disadvantages of various designs of freight houses and shop floors; (3) present data on rest houses for employees.

Wooden Bridges and Trestles.—(1) Continue study of relative economy of repairs and renewals of wooden bridges and trestles; (2) report on design of docks and wharves; (3) report on developments in practice of ballast deck trestles since previous report; (4) report on use of lagscrews for fastening guard timbers.

Iron and Steel Structures.—(1) Report on methods of protection of iron and steel structures against corrosion; (2) study designs and report on built-up columns, cooperating with other investigators and committees of other associations; (3) report on design, length and operation of turntables; (4) report on relative economy of various types of movable bridges.

Masonry.—(1) Complete report on principles of design of plain and reinforced retaining walls and abutments; (2) collect data concerning cost and method of constructing concrete piles, and make recommendations as to their use; (3) report on cost, appearance and wearing qualities of various methods of surface finish for concrete.

Signs, Fences and Crossings.—(1) Report on economy of concrete and metal signs and signals as compared with wood; (2) report on economy of concrete and metal as compared with wood for fence posts; (3) investigate methods used and comparative cost of repainting crossing and other signs; also present specifications for whitewashing cattle-guard wing fences.

Signals and Interlocking.—(1) Continue study of economics of labor in signal maintenance; (2) formulate and present requisites for switch indicators, including conveying information on condition of block to conductor and engineman; (3) present, for approval, specifications adopted by the Railway Signal Association, which, in the judgment of the committee, warrant consideration; (4) study problem of signaling single-track roads with reference to effect of signaling and proper location of passing sidings on capacity of line.

Rules and Organization.—(1) Review rules and instructions heretofore adopted and recommend such changes and additions thereto as may seem desirable; (2) formulate rules pertaining to safety for guidance of maintenance-of-way department; (3) continue formulation of rules for guidance of field parties: (a) When making preliminary surveys, (b) when making location surveys, and (c) when in charge of construction; (4) continue study of science of organization.

Records and Accounts.—(1) Make comprehensive study of forms adopted a number of years ago, and bring up to date; (2) continue study of reports required by Federal and State railway commissions; (3) continue study of feasible and useful subdivision of Interstate Commerce Commission classification account 6, with a view to securing uniformity of labor costs.

Uniform General Contract Forms.—Continue study of general contract forms, including forms for bond.

Water Service.—(1) Complete report on design and relative economy of track pans from an operating standpoint; (3) report on deep wells and deep-well pumping and relative economy as compared with other sources of water supply; (3) report on use of compounds in locomotive boilers to counteract: (a) Foaming and (b) scale formation; (4) continue study of recent developments in pumping machinery, and various kinds of fuels used.

Yards and Terminals.—(1) Report on typical situation plans of passenger stations, of both through and stub types, with critical analysis of working capacity, and include a review of the different methods of estimating their capacity; (2) report on developments in handling of freight by mechanical means; (3) report on developments in design and operation of hump yards; (4) continue study of track scales.

Economics of Railway Location.—(1) Study question of grade, curvature, rise and fall, and distance, and, if possible, present conclusions as to reasonable values of same in usable form, in order that they may be of use for the information and guidance of locating engineers; (2) continue important study of economics of railway operation heretofore carried on by the committee, in order that the information may lead to more economical methods in railway operation and that information may be obtained for correcting values given to the physical features in the locating of railways; (3) make special efforts to collect information in regard to effects of passenger and freight traffic on cost of maintenance.

Wood Preservation. — (1) Continue study of use of coal tar in creosote oil; (2) continue compilation of available information from service tests, supplementing this with reports of inspections to be made by members of the committee, of those sections of test track that have been in service long enough to give results; (3) present specifications for timber to be treated; (4) report on methods of accurately determining absorption of creosote oil; (5) study subject of water in creosote.

Electricity.—(1) Continue study of clearances; (2) report on effect of electrolytic action on metallic structures, and best means of preventing it; (3) continue preparation of standard specification for overhead transmission-line crossings; (4) continue study of electrolysis and insulation and its effect upon reinforced-concrete structures; (5) report on maintenance organization with relation to track structures.

Conservation of Natural Resources.—(1) Continue study of tree planting and general reforestation; (2) continue study of coal, fuel, oil and timber resources; (3) continue study of iron and steel resources.

Grading of Lumber.—Continue collection of current specifications and grading and inspection rules for maintenance-of-way timber and lumber not heretofore reported on, and present same for all classes of maintenance-of-way timber and lumber, which will conserve the interests of railways and be acceptable to manufacturers' associations—conferring with other committees of the association and with other organizations whose work is affected.

PORTLAND CEMENT works with an annual capacity of 120,000 tons have been built at a cost of $700,000 at Aberthaw, South Wales, according to a recent consular report. The works have tidewater communications. Adjacent to the works are quarries of argillaceous clay and limestone, said to be of the same quality as the deposits in the Lehigh Valley, Pennsylvania.

"Engineer" Defined in British Columbia Water Act

Successful Result of Efforts of Canadian Society of Civil Engineers to Get Recognition from Provincial Government

THE British Columbia members of the Canadian Society of Civil Engineers, as announced in this journal March 7, 1914, page 274, have recently shown considerable activity in endeavoring to get the status of the engineer recognized by the Provincial Government, and in view of much new legislation, involving large engineering projects, several influential deputations of members, headed by F. C. Gamble, chief engineer of the department of railways, and G. R. G. Conway, chief engineer of the British Columbia Electric Railway Company, Ltd., the respective chairmen of the Victoria and Vancouver branches, met the Premier, Sir Richard McBride, and the executive council and laid before them very strongly the views of the Western members.

It was pointed out that in numerous acts of Parliament the definition of an engineer was extremely unsatisfactory. One example given was quoted from the Ditches and Water Courses Act of 1907, where an engineer is defined as follows: "Engineer means civil engineer, Provincial land surveyor, or such a person as any municipality may deem competent and appoint to carry out the provisions of this act." The effect of this definition in one instance in British Columbia was the temporary appointment by resolution of the municipal council of a small contractor to sign plans, and the deputation pointed out to the ministers that there was nothing in the act to prevent a municipal council from appointing a gardener, blacksmith or carpenter to carry out the duties of the act as engineer.

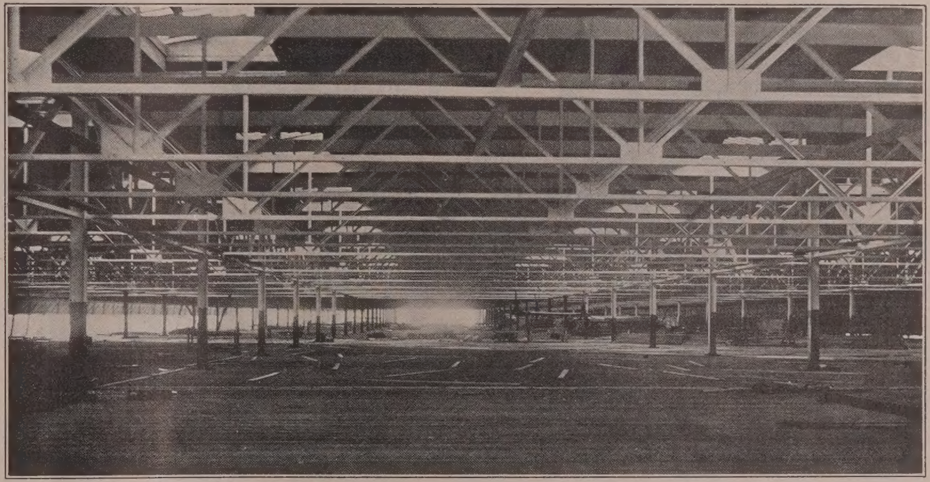
The deputations were accorded a very sympathetic hearing from the members of the Government, and many private members supported their proposals. Subsequent to the interviews with the Government a petition was sent to the premier signed by every leading engineer in the province, asking that the word "engineer" be defined in the new Water Act as follows:

"Engineer means an engineer of the water district, who shall be a civil engineer who is a member of the Canadian Society of Civil Engineers, or a member of the institutions of civil engineers of Great Britain or of Ireland, or a member of a civil engineering society of equal rank in the British Empire or any foreign country."

This definition was accepted by the minister of lands, the Hon. W. R. Ross, who, however, felt that while entirely recognizing the principle of the definition, and expressing himself in hearty accord with the objections of the members of the Canadian society, thought that special circumstances might arise in which he might be hampered by too strict an observance of the definition. He, therefore, suggested the addition, which was mutually agreed upon, of the following words, that now, together with the foregoing, form part of the Water Act of 1914:

"But the foregoing definition shall not limit the power of the Lieutenant Governor in Council or the Minister to appoint to any position created under this Act any person who is in his opinion competent to fill such position."

The Western members feel that they are to be congratulated upon the success of their efforts and hope during the next session to have a similar definition incorporated in the Municipal Act, the Railway Act, and other acts dealing with engineering undertakings.



Spacious Second Floor of New Pier at Bush Terminal, Brooklyn

Large Double-Deck Pier at Bush Terminal in Brooklyn

New Structure 1400 Feet Long and 270 Feet Wide, with Floor Area of 15 Acres

A DOUBLE-DECK pier 1400 ft. long, 270 ft. wide, with a total floor area of 15 acres, has just been completed at the Bush Terminal in Brooklyn, at a cost of $1,000,000. The pier is expected to accommodate six transatlantic steamers at one time and is so designed that unloading to the upper and loading from the lower floor can be carried on simultaneously. The broad expanse of floor space is shown by the view of the upper deck. In the center of the lower deck are two tracks connecting with the other tracks of the Bush Terminal. Each floor has 23 ft. of headroom. The lower floor is paved with fireproofed asphalt block. The upper floor consists of 1-in. yellow pine on 4-in. planking, with a top layer of 1-in. maple, and with intermediate layers of tar paper and melted tar to make the floor airtight and waterproof. The structure has a steel frame, as the illustration shows, and is carried on wooden piles. Like all other Bush Terminal buildings and piers, it has an automatic sprinkler system.

Frictional Resistance in Artificial Waterways

Summary of Investigations Made to Obtain Experimental Coefficients for Flow in Open and Closed Channels

A SERIES of field experiments was conducted in Colorado during the irrigation seasons of 1912 and 1913 in order to add to the available information on proper values of the coefficient of roughness in various types of artificial waterways. The principal types of channels experimented upon were metallic, reinforced-concrete and timber flumes, concrete-lined canals, earth canals, concrete and timber chutes, and inverted siphons of wood-stave pipe, located on irrigation systems in the Cache la Poudre, Arkansas, Grande, Gunnison, Uncompahgre and San Luis valleys. The results of these investigations have been published in Bulletin 194 of the Agricultural Experiment Station of the Colorado Agricultural College. The work was carried out by V. M. Cone, R. E. Trimble, and P. S. Jones.

Values for the coefficient of roughness in the general formula of Kutter were secured from the field data by graphic methods for slopes up to 0.0621 ft. per foot and by computation for greater slopes.

It was found that the weight of the water in metallic flumes caused distortion of the original semicircular cross-section, so that it was necessary to make two sets of measurements to obtain the wetted cross-sectional area and wetted perimeter. According to the characteristics of their interior surfaces, the flumes were divided into three classes: (1) Those whose connections at the joints are countersunk to the plane of the sheet metal, and which present a smooth, unobstructed water face; (2) those whose joint connections protrude into the waterway beyond the plane of the sheet metal, and (3) those constructed of corrugated metal and whose corrugations at right angles to the line of flow offer the only frictional resistance.

Six timber flumes were experimented upon. Some of them were more than twenty years old, and one was lined with a patented roofing material. Concrete-lined and earth channels were also investigated. Table 1 gives a summary of the coefficient of roughness (n).

SIPHONS

A NUMBER of wood-stave inverted siphons were also investigated. Tabulated results are given in Table 2. The high values of n obtained for some of the siphons indicates a deposit of débris at the low point of the siphon or other conditions retarding the flow of water. It is, therefore, not to be understood that values for n of 0.027, or even 0.019, are recommended for wood-stave siphons properly constructed and in good condition of alignment and interior. The siphons which were expected to be in good condition did show low values of n, but the entire data indicate the condition that may be obtained in siphons after a few years of operation where no provision is made for cleaning them.

Oftentimes in determining the carrying capacity to be assigned to a channel under construction doubt arises as to the proper coefficient of roughness to use, or as to what safety factor to apply for any possible future variation in the value of the coefficient.

TABLE 1—SUMMARY OF COEFFICIENT n

	n
Semicircular Metallic Flumes	
Smooth, unobstructed water face; with wasteways, and of length less than 100 ft.	0.021
Smooth, unobstructed water face..........	0.011
Joint connections protruding into waterway beyond plane of sheet metal............	0.0174
Corrugations at right angles to line of flow..	0.0225
Timber Flumes	
Lined with patent roofing material.........	0.016
Planed lumber, battens on sides; transverse flooring	0.015
Planed lumber; battens on the sides; longitudinal flooring	0.012
Planed lumber; no battens; perfect alignment	0.011
Concrete-Lined Channels	
Smooth concrete or mortar-finished surface.	0.012
Surface left unplastered in condition given it by forms	0.016
Earth Channels	
Bed exceptionally smooth and of fine silt; very uniform in cross-section; perfectly straight alignment	0.017
Bed of hard, water-worn adobe............	0.022
Bed of coarse gravel and small, loose stones.	0.024
Banks of smooth clay; bottom of fine sand; grass overhanging banks into water......	0.027
Bed of hard, water-worn adobe; many fine roots projecting	0.028
Bed of fine silt, merging into clay; liberally sprinkled with large, loose stones........	0.030
Bed of little gravel, but composed mainly of smooth, rounded stones, 6 in. or more in thickness	0.032

Table 3 illustrates the manner in which the discharge fluctuates with a variation in n. Four typical classes of lining were selected to show this—metal, concrete, timber and earth. The middle row of each group is from data actually observed in the field, while the others are computed from values of n changed by five units in the fourth decimal place, the hydraulic radius and slope remaining constant in all cases. From the percentage values which refer to each group it will be observed that the discharges vary in the inverse ratio as the coefficient of roughness, the hydraulic radius and slope remaining constant.

TABLE 2—HYDRAULIC ELEMENTS OF WOOD-STAVE SIPHONS

Diameter, in.	Length of pipe, ft.	Maximum pressure head, ft.	Discharge sec. ft.	Mean velocity, ft.	Total loss of head, ft.	Loss of head per 1000 ft.	c*	n†	f‡
48	486	51	20.34	1.62	0.497	1.021	49.4	0.0271	0.1005
48	515	66	20.44	1.63	0.202	0.392	82.2	0.0172	0.0380
36	815	..	15.25	2.16	0.515	0.632	98.6	0.0140	0.0262
60	363	37	49.70	2.53	0.669	1.843	52.8	0.0275	0.0925
60	1621	99	49.70	2.53	1.045	0.644	89.7	0.0169	0.0324
60	426	55	50.73	2.58	0.596	1.400	61.8	0.0238	0.0675
60	323	26	53.99	2.75	0.368	1.141	72.7	0.0204	0.0486
60	405	59	53.99	2.75	0.408	1.007	77.6	0.0193	0.0429
58	2278	104	52.68	2.88	1.162	0.510	116.0	0.0134	0.0192
54	544	83	52.68	3.31	0.942	1.732	75.0	0.0195	0.0458
54	2563	60	53.99	3.39	2.205	0.861	109.1	0.0139	0.0220
54	803	77	53.99	3.39	1.438	1.790	75.6	0.0193	0.0449

*Value of c in Chezy formula $V = c \sqrt{rs}$.
†Value of n in Kutter formula.
‡Value of f in formula for lost head due to friction and curvature, $h = f\ (L\ V/2\ g\ d)$

In this connection there may also be noted the accuracy of current-meter measurement required to produce an erroneous value of n. For a low value of n a 4-per cent error in current-meter measurement changes the coefficient 0.0122 to 0.0117, for instance. Or, with high value of n a 2-per cent error in meter measurement changes 0.0285 to an erroneous value of 0.0280. Hence it is believed an error of 1.5 per cent, or less, in current-meter measurement was allowable in obtaining the correct discharge of the various channels under test.

TABLE 3—EFFECT UPON n OF VARIATION IN HYDRAULIC FACTORS

Type of channel	Comparative per centages of Q	Discharge, sec-ft.	Area of wetted section, sq. ft.	Velocity, ft.	Wetted perimeter, lin-ft.	Hydraulic mean radius	Slope per ft.	c	n	Comparative percentage of n
Metallic flume...	103	34.45	5.77	5.97	6.30	0.916	0.00537	85.05	0.0170	103
	100	33.30	5.77	5.77	6.30	0.916	0.00537	83.21	0.0175	100
	97	32.26	5.77	5.59	6.30	0.916	0.00537	79.65	0.0180	97
Concrete chute...	104.5	128.92	6.24	20.66	7.24	0.862	0.02971	128.80	0.0117	104.5
	100	122.94	6.24	19.70	7.24	0.862	0.02971	122.82	0.0122	100
	95.5	117.48	6.24	18.83	7.24	0.862	0.02971	117.36	0.0127	95.5
Timber flume....	104	228.76	36.79	6.22	18.12	2.030	0.000903	145.28	0.0117	104
	100	219.40	36.79	5.96	18.12	2.030	0.000903	139.32	0.0122	100
	96	210.88	36.79	5.73	18.12	2.030	0.000903	133.92	0.0127	96
Earth canal.....	102	720.61	146.05	4.93	65.70	2.222	0.00307	59.59	0.0280	102
	100	707.00	146.05	4.84	65.70	2.222	0.00307	58.45	0.0285	100
	98	694.61	146.05	4.76	65.70	2.222	0.00307	57.44	0.0290	98

Ottawa's Water Controversy

Conclusion Reached after Investigation by Experts of Three or More Different Projects

AS announced in this journal April 4, Ottawa, Canada, has concluded to use the Ottawa River as a source of water supply, purifying it by rapid sand filters.

Ottawa has had advice from recognized experts on at least three different projects. All of them have been voted on and defeated for one cause or another. The present supply is taken from the Ottawa River without filtration and is conveyed to the city through pipes under the river and a power canal, in which the water is contaminated. Leakage into the intake pipes has caused typhoid epidemics. The Ottawa River itself, as compared with other streams from which water for domestic purposes is secured, is comparatively free from pollution, but it is highly colored, soft and contains large quantities of organic matter. Slow sand filtration was popular in Ottawa, but its use has been advised against by all except one of the experts who have reported upon it, and has been prohibited by the Provincial Board of Health.

In 1910 Allen Hazen reported to the city, recommending a supply from McGregor Lake, 15 miles north of the city. The catchment area is about 94 sq. miles, and Mr. Hazen estimated that 54,000,000 imp. gal. per day could be obtained from this source at a cost of about $2,300,000. As an alternative he proposed the filtration by rapid sand filters of Ottawa River water. Following the report a commission was appointed consisting of Mr. Hazen, Dr. Hodgetts, Dr. McCullough and C. H. Keefer. The first three members recommended the use of McGregor Lake. Mr. Keefer dissented, stating that the information as to the quantity of water available was insufficient to justify the city in adopting the McGregor supply. He recommended that the use of Ottawa River be continued. The project to install the McGregor Lake works was, however, defeated in 1911 on the basis brought forth by Mr. Keefer.

Then the City Council directed the city engineer to draw plans for slow sand filters to handle the Ottawa River water, despite the statement by Mr. Hazen, City Engineer Ker and others that the system was not adapted to the filtration of Ottawa River water. These plans were submitted to the Provincial Board of Health, which refused to approve them. After a second typhoid epidemic the Provincial board ordered the city to proceed forthwith with the installation of a rapid sand filtration plant along the general lines laid down by Mr. Hazen in his report of 1910. Messrs. Hazen and Whipple were selected to design the plant, and plans were made for these filters; but before proceeding with the work it was necessary to secure the approval of the people and a vote was taken in January, 1913. The well-known cry of "doped water," "corrosion of pipes" and other slogans were used by the daily papers with telling effect. Rapid sand filtration was overwhelmingly defeated, due largely to a mayoralty candidate's announcement that if the rapid sand filter project were killed he would secure a British engineer to report on the water supply.

Report of British Engineers

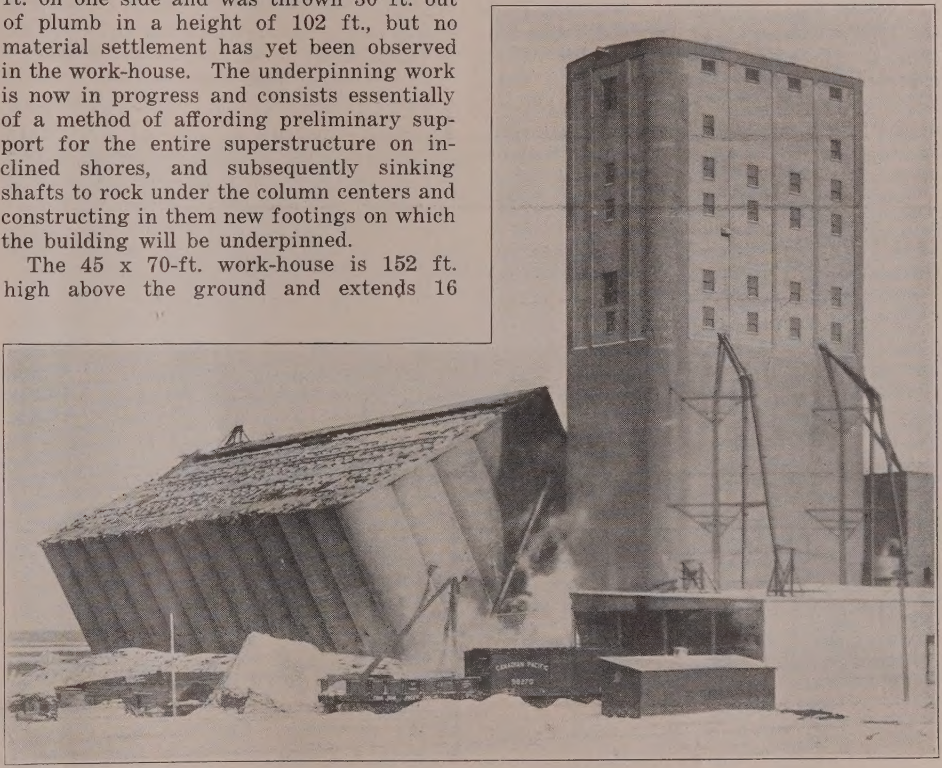
PROCEEDING through official sources, the mayor secured Sir Alexander Binnie and Dr. A. C. Houston, of London, to advise on the situation. They came to Ottawa in February, 1913, and in a tentative report recommended the adoption of the northern lakes, without selecting any particular one of them. The final report, rendered in October, 1913, recommended that a supply be obtained from Thirty-One Mile Lake, about 50 miles from Ottawa, at a cost of $8,000,000.

This report was accepted by the council, which immediately proceeded to carry out the works. Opposition to the plan soon developed and the matter was brought to the courts. The ensuing fight resulted in a series of defeats for those who favored the adoption of the project without a vote of the people. A bitter controversy over the relative merits of the different schemes developed. It was claimed by the opponents of Thirty-One-Mile Lake that Sir Alexander Binnie's report was misleading. They brought out the fact that although his final recommendations stated that 81,000,000 gal. could be obtained for $8,000,000, the report itself showed that the estimate was based upon pipe lines of a capacity sufficient only for a 25,000,000-gal. supply. The Binnie report increased the Hazen estimate for McGregor Lake to $7,000,000, including in this figure not only $500,000 for filters but also $2,000,000 for the acquisition of the watershed and a large increase in the cost of pipe lines so as to include in the project a reservoir located on a site selected for the Thirty-One-Mile Lake scheme. The Thirty-One-Mile Lake project, like the others, went down in defeat. Now by popular vote, polled on March 30, the use of Ottawa River water, mechanically filtered, has been approved. The plan adopted is the alternative originally proposed by Mr. Hazen in 1910.

Underpinning Work-House Adjacent to Tilting Grain Elevator at Transcona

To Prevent Settlement Columns of High Reinforced Concrete Building Were Supported by Inclined Shores while New Footings Were Built

TO forestall the settling of the tall, heavy work-house adjacent to the Canadian Pacific Railway's grain elevator at Transcona, Canada, which is leaning at an angle of 24 deg. to the vertical as a result of recent subsidence of the blue clay foundation, a contract for underpinning has been awarded to the Foundation Company, Ltd., of Montreal. The main elevator settled 26 ft. on one side and was thrown 30 ft. out of plumb in a height of 102 ft., but no material settlement has yet been observed in the work-house. The underpinning work is now in progress and consists essentially of a method of affording preliminary support for the entire superstructure on inclined shores, and subsequently sinking shafts to rock under the column centers and constructing in them new footings on which the building will be underpinned.



Underpinning Work House Adjacent to Transcona Elevator

The 45 x 70-ft. work-house is 152 ft. high above the ground and extends 16 ft. below ground to the top of a monolithic concrete foundation slab 4 ft. thick which is heavily reinforced and receives the square footings of twenty-four cylindrical columns arranged in four longitudinal rows. The estimated total weight of the building and contents is 5640 tons. The interior columns each carry 313 tons live load and 415 tons dead load, while the exterior columns carry 157 tons live load and 215 tons dead load. The foundations rest on a bed of clay about 40 ft. thick, below which there is a stratum of gravel and disintegrated rock extending to bed rock about 54 ft. below the surface.

The unstable condition of the soil, demonstrated by the excessive displacement of the adjacent building, made it very hazardous to disturb these foundations until independent support was provided for the superstructure. The entire weight of the building was therefore transferred to temporary new foundations through a system of inclined shores on exterior piers with a factor of safety of three.

A sheeted trench, heavily braced with 12 x 12-in. horizontal timbers, was excavated around all four sides of the building to a depth below column footings, and in it twenty foundation pits were sunk to rock opposite the exterior columns and about 10, 13 and 15 ft. distant from them on centers. The two pits at each corner, eight in all, were excavated inside a sheeting of sectional cast-iron cylinders with courses 4 ft. high, which were loaded with steel rails suspended from an upper platform.

The experience and information obtained by sinking these pits enabled the contractor to sink the remaining pits, 5 and 6 ft. in diameter, by the Chicago method, substituting for the cast-iron casing vertical wooden staves with inside flanged steel rings. The piers on the longitudinal sides of the building were filled with 1:2:4 concrete and the larger ones at the ends were filled with 1:3:5 concrete. They were capped with enlarged rectangular pedestals to receive the feet of the inclined shores. The thrust was resisted by horizontal rails embedded in the concrete and bearing against vertical rails reinforcing the top of the pier and distributing the stress into the pier shaft.

Transferring Column Loads

THE column loads were all transferred to the new exterior piers by a system of heavy multiple inclined shores transverse to the longitudinal axis of the building. These shores were arranged in pairs, one shore of each pair engaging the adjacent exterior column at the second floor, and the other engaging the adjacent interior column in the same transverse plane at the second floor level. The corresponding pair of shores in the same plane on the opposite side of the building completed the support for one transverse row of columns, and with the solid floor connecting the columns formed an arch-like construction through which the total weight of the upper stories was transferred to the new outside piers. Provision was made for the insertion of additional bracing from the bottoms of the exterior columns to the first floor, as indicated by dotted lines, but their use has not been found necessary. The tops of the exterior foundation piers and some of the footings of the columns in the building are braced by raker struts to the bank on the outside of the trench.

UNDERPINNING

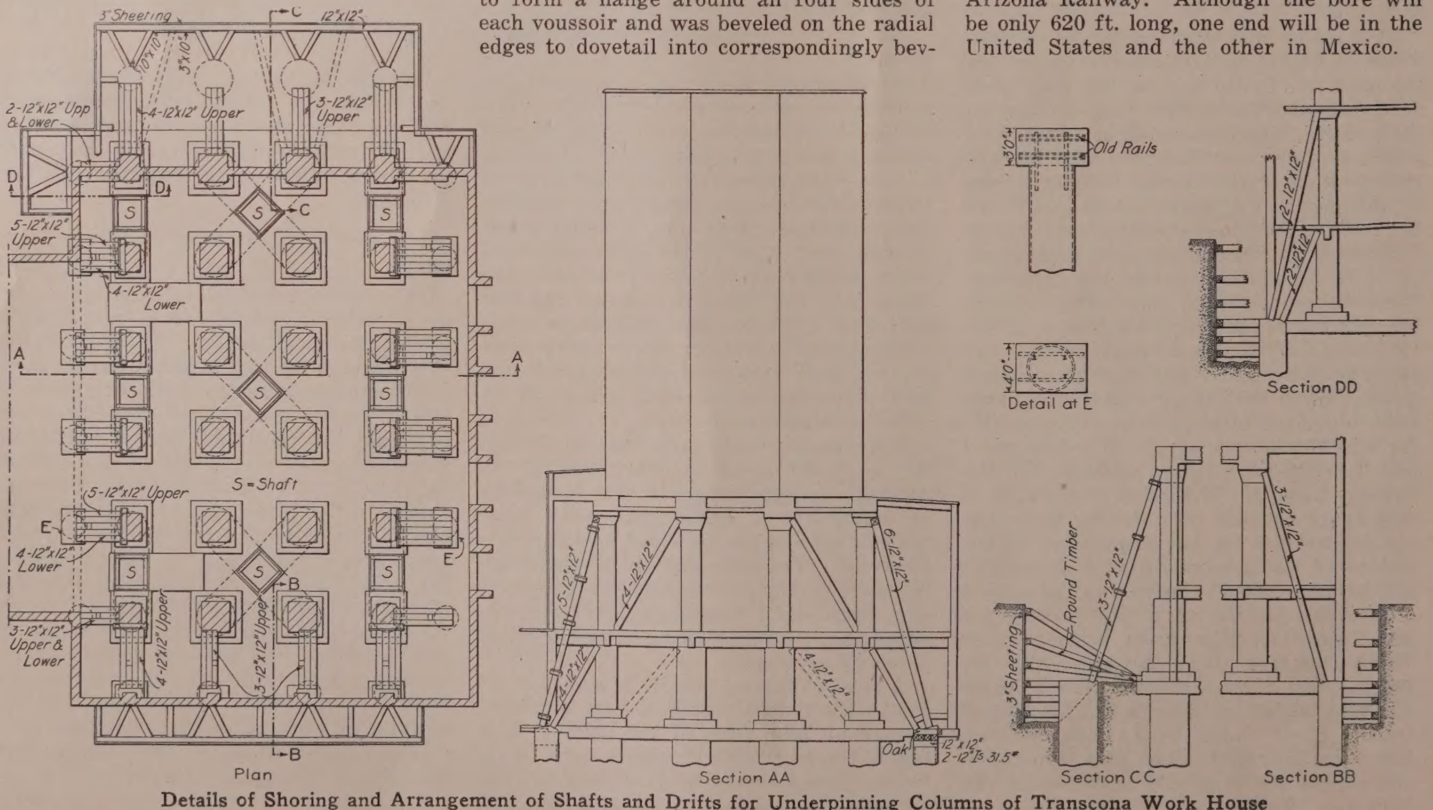
THE shores are made of full length 12 x 12-in. timbers, from two to four in a set. In the heaviest long shores the separate pieces lie in the same inclined plane and are thoroughly clamped together with yokes made of a pair of screw rods with nuts engaging bearing angles on the opposite sides of the struts. The upper ends of the outside two-story shores engage horizontal bearing pieces or caps consisting of 12 x 12-in. timbers 6 ft. long reinforced by a pair of 12-in. channels fitted on opposite faces and securely bolted to them. These pieces are set in seats cut out of the concrete at the tops of the columns, and distribute the shore pressure over the entire width of the concrete. The other shores have their ends bearing directly against the dressed surface of the concrete. The shores were adjusted and brought to bearing by heavy oak wedges at the lower ends.

The monolithic concrete slab which forms a continuous footing under the entire area of the building will have nine openings cut through it to give access for the underpinning excavation. These openings, about 6 ft. square, will be located so as to give access to the greatest number of columns without involving unnecessary cutting of the foundation slabs. Three of them will each give access to four interior columns, and the outer ones will each give access to two exterior columns.

From these openings excavations will be sheeted down about 6 ft. below the bottom of the concrete and drifts will be run from them to the centers of the adjacent columns under which shafts will be excavated by the Chicago method already described, carried down to rock, and filled with concrete, adjusted to bearing under the column bases so as to continue the support of the latter directly to solid bearings.

Each shaft will be equipped with a small individual electric hoist, and the spoil buckets will be suspended from trucks engaging horizontal tracks on which they will be carried through the drifts to the common opening in the foundation slabs. In the excavation of the exterior shafts for the shoring piers, the spoil buckets were handled by windlasses operated in groups by a rope drive.

The work is being executed under the supervision of A. I. Campbell, superintendent for the contractors, and Frank Lee, principal assistant chief engineer in charge of the work for the Canadian Pacific Railway Company.



Details of Shoring and Arrangement of Shafts and Drifts for Underpinning Columns of Transcona Work House

---

Long Cast-Iron Arch Spans

Details of the Old Southwark Bridge over the Thames River, Now Being Replaced after Ninety-Five Years' Service

REPLACEMENT by a modern steel truss structure of the old Southwark bridge across the Thames in London, which is now in progress, calls attention to the character and efficiency of the old structure, which had been in service for ninety-five years, and which is briefly described in a recent issue of "Engineering."

The bridge was designed by Sir John Rennie and had one 240-ft. center span flanked by two 210-ft. side spans carried by masonry piers on timber platforms with wooden-pile foundations. It had a width of 42½ ft. between parapets, and the cast-iron ribbed floor plates, 1¼ in. thick, were supported on cast-iron latticed spandrel girders seated on eight lines of arch ribs spaced about 6 ft. 3 in. on centers. The bridge had a rise of 23 ft. 9 in. for the center span and 18 ft. 3 in. for the side spans.

KEYED VOUSSOIRS

THE ribs were made of voussoirs about 17 ft. long, 6 ft. deep and 2½ in. thick at the crown and 8 ft. deep and 3 in. thick at the skewback. The web was stiffened to form a flange around all four sides of each voussoir and was beveled on the radial edges to dovetail into correspondingly beveled flanges cast integral with transverse radial diaphragms, of the full depth of the voussoirs, that extended continuously across the entire width of the bridge. The voussoirs were locked into the diaphragms by full-length wedges, tapered slightly, and each diaphragm was made in two pieces, spliced in the bridge axis by a vertical web joint with a wedge key engaging the pairs of interlocked flanges. Some of the panels between ribs were braced by horizontal tapered lateral diagonals with cruciform cross-sections having their bent ends bolted to the voussoir webs.

The weight of the diaphragms was reduced by large rectangular openings in each panel, and their upper flanges were grooved to engage tongues of the lower edges of the spandrel girders. The principal connections were made with the wedges, reducing the number of connecting bolts to a minimum. All material was cold-blast cast iron of exceptionally good quality.

The bridge was erected with all voussoirs assembled on folding wooden camber wedges. After the ribs in all three spans were completely assembled three pairs of 6 x 9-in. permanent iron skewback wedges 9 ft. long were inserted between each end of each rib and the masonry bearing, and were approximately adjusted with their lower ends 12 in. above the bottoms of the ribs. All were simultaneously driven about 13 in. by wooden rams, gradually bringing the three spans to bearing without raising them completely from their camber wedges. They were allowed to remain in this position for a few days, during which a rigid inspection failed to reveal any signs of imperfection. The camber wedges were then slacked off, causing a deflection of about 2½ in. in the center span and 2 in. in the side spans.

AN INTERNATIONAL TUNNEL, said to be the only tunnel in North America to cross an international boundary line, is about to be started on the San Diego & Arizona Railway. Although the bore will be only 620 ft. long, one end will be in the United States and the other in Mexico.

Concrete Road Crossing a Flood Plain

One-Mile Highway on Old Macadam Surface Along Steep Embankment Near La Salle, Illinois—Concrete Guard Rails and Light Fixtures

WIDELY varying problems of design and construction were encountered in building a concrete road, 1 mile long, across the bottom land of the Illinois River in La Salle Township, Illinois. A. H. Hunter, division engineer, State Highway Commission, described the features of the improvement at the annual meeting of the Illinois Society of Engineers and Surveyors in a paper from which the following notes have been taken.

For three-quarters of its length the road is subject to the ravages of the stream in flood time; the other fourth is on the bluff approach to La Salle from the south, passing under the Burlington Railroad viaduct, up a 6-per cent grade over the Illinois-Michigan Canal and up another grade—5.5 per cent—to a bridge over the Rock Island Railroad tracks.

The flood-plain section is laid on an old waterbound macadam surface not more than 22 ft. wide on top, with steep slopes on either side extending to the bottom land 10 or 12 ft. below. The other part, with shale and cinder surface, was restricted in width by the Burlington viaduct, and on the 6-per cent grade was only 18 ft. wide on top. Even then the toe of the side slope extended into the dooryards of adjacent property. Traffic consists of many heavy trucks and a number of pleasure vehicles.

At the instance of the La Salle Commercial Association, the State Highway Commission made survey and plans for a 20-ft. roadway 6 in. thick at the sides and 7 in. at the center, with a crown of $3\frac{3}{4}$ in. Construction was by day labor, except hauling, under the supervision of the highway commission, with Mr. Hunter in charge. The commission also furnished free a 10-ton road roller, a concrete mixer and an operator for each.

Drainage for the foundation was provided by longitudinal ditches 8 in. wide and 6 in. deep under the edges of the metal. At the end of expansion joints a lateral drain, consisting of a trench filled with gravel, leads to the riprap wall at the downstream side of the fill.

Concreting on Old Macadam

THE old macadam surface was loosened with a scarifier drawn by the roller; the loose stone was then shaped to the crown of the subgrade and rolled. The surface, which contained an abundance of stone dust, was thoroughly wetted, taking a hard, smooth finish which gave an excellent foundation, as well as furnishing a smooth surface on which to handle the gravel.

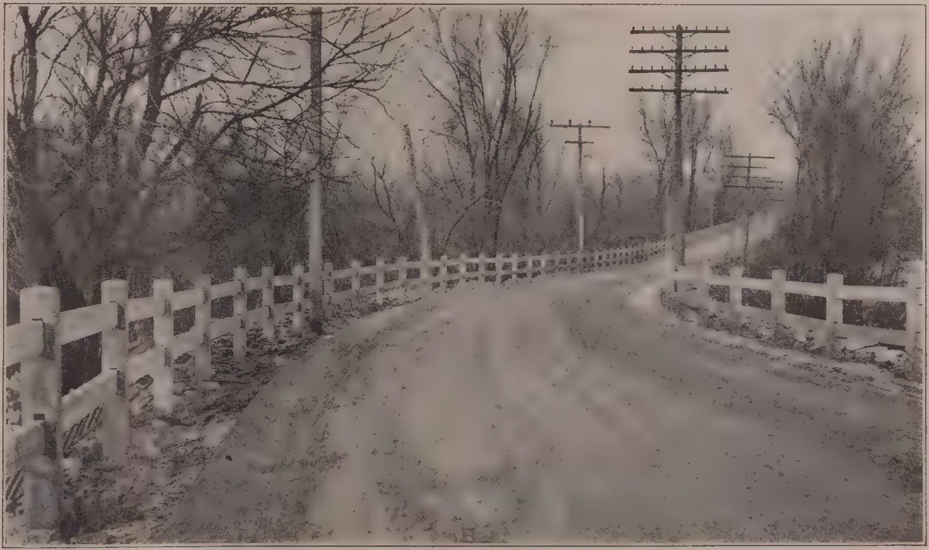
The aggregates were screened, washed gravel—granite pebbles—sized from $\frac{3}{8}$ to $1\frac{1}{4}$ in.—and clean, rather fine sand. On the rolled subgrade the loads of gravel and sand were dumped in rows. The quantities per linear foot of road were computed and the wagons measured and loads spaced accordingly. Ten per cent excess of material was provided to allow for waste and variation in thickness. This amount proved sufficient, as there was not an excess or deficiency of more than 10 cu. yd. of sand and gravel in the whole road.

All cement used, comprising some 3200 bbl., was donated by the Marquette Cement Manufacturing Company and the Chicago Portland Cement Company. The uniformity of the mix—$1:2:3\frac{1}{2}$—was tested frequently with a wooden measure holding one bag of cement.

Provision for the large amount of water needed for this type of road, both for mixing and curing, was made by laying a $1\frac{1}{4}$-in. pipe from the city system, with T's every 300 ft.

A self-traction batch mixer was used, with a low hopper in front into which sand and gravel were shovelled direct from the road bed. Two sacks of cement were used for the batch. The mixed material was deposited on the subgrade by means of a 20-ft. rotating boom on which operated a movable bucket. The number of square yards laid per day of eight hours averaged 450.

The soft concrete was struck off to crown and grade with a steel-shod template. Expansion and contraction were taken care of by means of $\frac{3}{8}$-in. asphalt joints placed 50 ft. apart at an angle of 60 deg. with the center line of the road. The joints were protected by Baker plates.



Concrete Roadway on Embankment across Valley at La Salle, Illinois

The guard rails are also of concrete, as are the brackets—light fixtures—attached to the concrete poles at the left.

The edges of the concrete were retained by wood forms consisting of 2 x 6 in. pine pieces, 16 ft. long, resting on edge, held to alignment by $\frac{5}{8}$-in. iron pins 18 in. long. It was found difficult to butt the boards together perfectly so the more imperfect form pieces were cut in 2-ft. lengths and spliced to the outside of the forms with small nails. This practice gave good results, especially on curves, where it was possible to align arcs of circles almost perfectly.

Care in curing the surface was taken in sprinkling and covering with canvas while setting, as is common, and in covering the finished surface with a layer of straw. No earth was available for this purpose, as the lower edge of the fill was swampy. The sprinkling continued ten days and the straw proved very efficient in retaining moisture.

Costs

DETAILED costs were figured from the daily expenditures, cost of hauling and the market valuation of the donated materials, including freight and incidentals. The salaries of the engineer and the operators of the roller and mixer were also added. The total cost on this basis, for 10,663 sq. yd., was \$13,632.16. Unit costs are given in the accompanying table.

The common labor used was foreign and received 35 cents per hour and teams received $62\frac{1}{2}$ cents per hour.

Concrete Light Fixtures and Fence

THE original plan included a 3-ft. concrete walk on the east side, but it was found that part of the walk would come upon a new fill, and the work has been postponed until the fill has become securely compacted.

The lighting system was adopted at the instance of a local utility company which offered to construct its poles with a special arm from which a light fixture could be suspended. Conduits were placed under the concrete every 120 ft. to provide connections for additional lights which will be added on completion of the sidewalk.

It had been planned to construct the guard fence of reinforced-concrete posts and two lines of 2-in. pipe rail. Later a series of experiments on the design of concrete fences was conducted by the Marquette Cement Manufacturing Company and one of the designs accepted. The successful design consists of a 6-in. square concrete post extending above the ground about $3\frac{1}{2}$ ft., reinforced by a $\frac{3}{8}$-in. rod placed near each corner. Slots secure two concrete panels—2 x $6\frac{1}{2}$ in.—reinforced by two small rods near the edges. A 1 x 1 x $\frac{1}{8}$-in. angle is embedded in the center of each rail, to prevent lateral bending.

The posts were placed 7 ft., center to center, and the concrete rails were slipped into the slots. This method of erection was quite simple, as no grout of any kind was necessary.

Costs per Square Yard for La Salle Concrete Road

Superintendence	\$0.023
Excavation, shaping road bed and trimming side roads	0.158
Hauling sand and stone	0.169
Mixing and placing concrete, setting forms and filling joints	0.133
Watchman and miscellaneous labor	0.116
Sand and stone, f.o.b. siding, including demurrage	0.314
Cement, f.o.b. siding, including demurrage	0.310
Expansion joints, f.o.b. siding, including demurrage	0.028
Coal and oil for mixer and miscellaneous supplies	0.021
Forms and other lumber	0.008
Total	\$1.280

Proposed New Mains to Avoid Water Shortage Due to Breaks in Cincinnati

Abstract of Report by S. G. Pollard, Recommending Construction of Three Miles of Sixty-Inch Conduit, One-Third in Tunnel

ON account of a shortage in the water supply in Cincinnati resulting from a break on Dec. 6, 1913, in the only pipe line, a 60-in. cast-iron main, through which water can be forced into the low-service district, much alarm was created by the serious risks which resulted. The city authorities, urged on by the fire-insurance interests, suggested the laying of a 30-in. pipe line around the city, a distance of some 13 miles, to connect with existing large feeders now supplied directly through the 60-in. main. This point of connection was to be about 6 miles from the business center of the city and about 10 miles from the main pumping station by way of the old mains. When the attention of the authorities was called to the fact that this 30-in. pipe line was too small, the size was increased to 36 in.

The 30-in. "loop main" through Linwood, Oakley, Norwood, Bond Hill and St. Bernard, five towns outside the city limits, had been discussed previously by the engineers of the commissioners of waterworks, at the time of the building of the new waterworks of the city, but only for the purpose of strengthening the distribution system in the eastern and northern part of the city. It was never discussed as a substitute for a second force main to carry water into the business section. For this latter purpose there had been proposed by the engineers for the commissioners a second 60-in. pipe line which was to convey water into Eden Park reservoir and which was to be laid as soon as conditions demanded it. For this purpose a 60-in. connection and a valve were provided at the main pumping station.

On account of the apparent confusion of the city, S. G. Pollard, consulting engineer and former superintendent of pumping, was retained by the city for the purpose of making a report on the suggested plan and on some other points relative to the pumping and filtration divisions of the water department. This report, which is comprehensive in its character, was submitted to the authorities on Feb. 9, but they have failed to act in accordance with its recommendations and have authorized a bond issue of $604,000 in addition to $96,000 in bonds already available for the purpose of constructing the "loop main."

EFFECT OF THE BREAK

THE break occurred at 6 a. m. Dec. 6 on Eastern Avenue just east of the Rockwood crossing of the Pennsylvania Railroad. The valves were closed, isolating the break in about two hours. Repairs were completed and water was turned on at 4 p. m. Dec. 10, four days and ten hours after the occurrence of the break. In the meantime one 10-in. and five 6-in. division valves between high and low service mains were wholly opened and one 8-in. valve was partly opened, beginning between 10 and 11 a. m. of Dec. 6 and continuing until 5 p. m. The next day, it having been determined that the valves already opened would not furnish a sufficient supply, pump valves were removed from one of the high-service engines at the Eden Park pumping station and the water fed back into Eden Park reservoir.

It was estimated that the loss in consumption in the low-service district was 22,600,000 gal. daily, or 12,700,000 gal. less than normal.

During the break the level of water in the east basin of the Eden Park reservoirs (the only one in service, the west basin having been out of commission for more than two years) fell from 30 ft. to 5 ft. and 3 in., the reservoirs being practically empty at the time the low-service pumping was resumed. That the fire danger must have become acute at this period is quite evident.

At the time of the break the low-service water pressure at the Western Hills station fell from 70 to 25 lb. per square inch, while the gage at the City Hall for a period of two hours registered only 5 lb. These pressures did not again become normal until midnight of Dec. 10.

FOUR REMEDIES STUDIED

FOUR possible methods of providing against a water shortage in case of a future break in the 60-in. low-service supply main were considered—first, Eden Park storage alone; second, bleeding the high-service and Eden Park storage; third, a 36-in. "loop" main plus the supplies in the second proposition, and fourth, an additional 60-in. main laid by the most direct route from the main pumping station into the city. The calculations and conclusions were based on 1913 and 1930 conditions, although data were furnished for years up to 1950.

Under the first proposition, with both reservoirs in commission, the low-service average daily rate of consumption could not be maintained for two days at the present time or more than one and a half days in 1930. Should the 60-in. main break during a period of highest daily consumption, the reservoir supply would, according to the 1913 figures, be exhausted in 1.4 days, while in 1930 it would last only 1.08 days. Experience has proved that with the present pipe connections it is absolutely impossible to maintain proper water service in the business sections, even with full reservoirs, without keeping at least one low-service pump in operation.

The second proposition is the identical experience through which the city just passed, and there was an actual shortage of 12,700,000 gal. per day in the low service, notwithstanding the fact that practically all of the stored water was consumed. Under average conditions of water consumption the supply in one reservoir would last three days and that in two reservoirs would last 6.4 days. Under conditions of maximum weekly draft one reservoir would maintain the service for 1.5 days and two reservoirs would keep up the supply for 3.2 days.

Project 3 involves the addition of 30, 36 or 42-in. "loop" mains, 69,700 ft. long. For the first 30,400 ft. from the main pumping station the proposed main can at present serve only 8000 people and only 12,300 in the year 1950. For the next 26,900 ft., from a point a mile north of Red Bank to the eastern boundary of the city of St. Bernard it will pass through territory too high for any services. For the next 7900 ft. the main must be laid through the streets of St. Bernard and the elevation will be such that services can be taken therefrom. The final 5500 ft. will be within the present boundaries of the city and will constitute practically an extension of the present 30-in. main in Spring Grove Avenue.

Based on cast-iron pipe twenty years old with $C = 100$, pressures of 75 lb. at the pumps and 55 lb. at the end of the line, the carrying capacities under the present consumption will be 6,930,000, 11,230,000 and 16,780,000 gal. for 30, 36 and 42-in. mains respectively. Under the consumption in 1950 these quantities will be 5,090,000, 9,390,000 and 14,940,000 gal. The 30-in. main does not provide enough water to consider it further, while with the 36-in. line, should a break in the 60-in. main in 1930 occur during a period of maximum consumption, the water in one reservoir would last only 1.5 days. The proposed loop main would be of use in maintaining the water supply of Cumminsville and the upper Mill Creek valley during a break in the principal force main, but good or even fair service could not be maintained in the business section of the city.

Theoretically a 42-in. loop main, over the route proposed, in connection with the water available from the high service and the reservoir service, should be able to maintain the low-service supply for a considerable period. Practically, however, it would be found that satisfactory service could not be maintained in the business district and that pressures would be low in many sections. Neither this nor the proposed 36-in. main could be considered as in any manner constituting a duplicate of the present 60-in. main.

By proposition 4, which calls for an additional 60-in. line, only can uninterrupted service be provided in case of a break similar to the one of last December, according to the report, and, furthermore, such a main will become a necessity in the course of a few years whether the "loop" main is constructed or not.

Objections to laying the new line on Eastern Avenue alongside the present main are that a break in one would cause failure of the other, and for considerable distances the existing line is below extreme high water, where at times repairs could not be made for two or three weeks. The route chosen is 12,600 ft. long, of which 5200 ft. will be in tunnel under Eden Park to connections in Gilbert Avenue near Elsinore Gate with mains leading from Eden Park reservoirs. Further extensions of 2900 ft. should be made to the city and to the site of the new proposed high-pressure fire-pumping stations.

Welded or lock-bar steel pipe of sheets ½ in. thick is recommended. Where it passes through the tunnel it should be entirely surrounded by concrete, and an additional outside protection of concrete is proposed for portions laid in the trench. Where slips have occurred in streets chosen it is proposed to sink it at least two-thirds of its diameter into shale rock for anchorage, making the center line depth from 10 to 12 ft. Preliminary investigation indicates, according to the report, that the construction of a 60-in. main is practicable and that it can be constructed for $679,790, less money than the proposed 36-in. "loop" main, but before any work is undertaken a complete survey should be made, test pits dug and drill holes bored along the tunnel route.

Locating Leaks in Water Mains by Means of Water-Hammer Diagrams

Paper Read before the Illinois Water Supply Association March 10 by M. L. Enger, Assistant Professor of Theoretical and Applied Mechanics, University of Illinois

When the valve at the end of a long pipe line is closed suddenly great pressures may be caused. The term water hammer has been applied to this phenomenon. If the valve could be closed instantly all of the water in the pipe would not be stopped at the same instant. The layer nearest the valve would stop first, then the next layer, and so on until the impulse has traveled through the entire pipe line. As each layer of water is brought to rest its pressure will, of course, be increased. The velocity of the transmission of the pressure wave will be the same as the velocity of transmission of sound in the water in the pipe, and will vary between 3400 and 4700 ft. per second, depending upon the material of the pipe and upon the ratio of the thickness to the diameter of the pipe.

It has been found that for any given pipe the amount of the water-hammer pressure is a constant times the extinguished velocity. The value of this constant (also called the water-hammer coefficient) varies directly with the velocity of transmission of the pressure wave, and for cast-iron pipe used for water supplies has values between 45 and 63. For cast-iron pipe between 6 and 16 in. in diameter, the average value of the constant is about 55—that is, the water-hammer pressure caused by the sudden closure of a valve at the end of a long pipe line is, in pounds per square inch, 55 times the velocity of the water in the pipe in feet per second, before the valve was closed.

Fig. 1a represents a pipe line in which there is a leak. The flow between the source (reservoir or large pipe) and the lock is $v + w$ ft. per second and between the leak and the valve, v ft. per second. Fig. 1b shows the conditions in the pipe line a short time after the valve at the end is suddenly closed. The velocity of the water near the valve has been extinguished and its pressure increased hv lb. per square inch (h being the water-hammer coefficient). If the distance from the valve to the leak is l ft. and the velocity of propagation of the pressure wave is Z ft. per second, the pressure wave will reach the leak l/Z seconds after the valve is closed. Since the original pressure at the leak allowed a quantity of water equal to Aw sec.-ft. to escape, it is evident that a higher pressure will cause a greater quantity to flow. The extinguished velocity between the leak and the source will therefore be less than v ft. per second. Hence the water-hammer pressure generated in this part of the pipe line will be less than hv lb. per square inch. A wave of reduced pressure will, therefore, travel from the leak toward the valve. Fig. 1c shows the conditions a short time after the pressure wave has passed the leak. The wave of reduced pressure will reach the valve $2l/Z$ seconds after the valve closed. Fig. 1d shows the conditions a short time after the wave of reduced pressure has reached the valve.

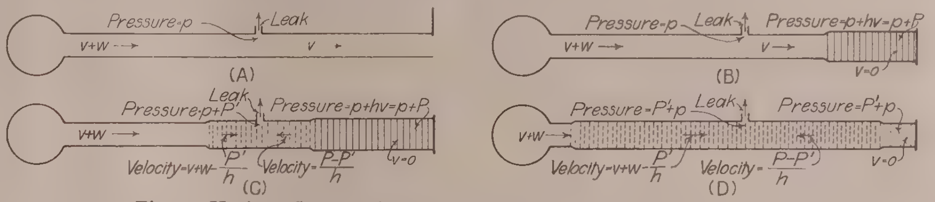


Fig. 1—Various Stages of Water Hammer in a Pipe Having a Leak

WATER-HAMMER DIAGRAM

The water-hammer diagram is a graphical representation of the pressure in the pipe line near the valve for a time after the valve is closed. In the experiments made by the writer the diagram is obtained by having the pencil of an indicator trace on a sheet of paper wrapped around a drum driven at a uniform rate by an electric motor. Another pencil attached to an electromagnet makes a time record. A photograph of the apparatus is shown in Fig. 2.

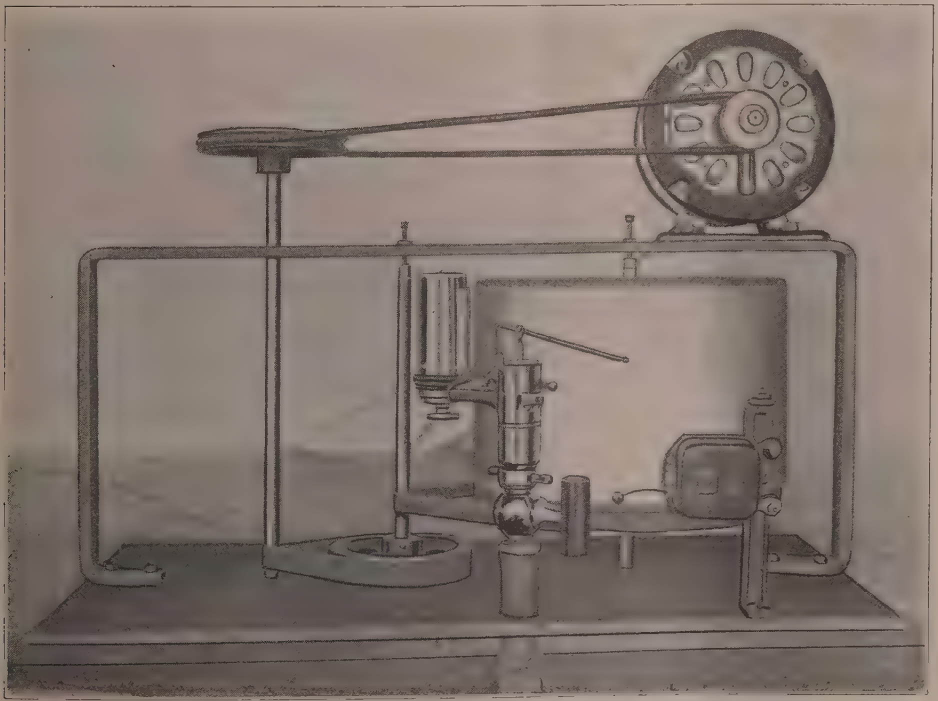


Fig. 2—Laboratory Apparatus Used for Water-Hammer Experiments

Fig. 3 shows the characteristic features of a water-hammer diagram taken at the end of a pipe line in which there is a leak. The first rise of pressure as the valve begins to close is shown at A. The indicator pencil reaches B when the valve is fully closed. The pressure then remains practically constant until the effect of the leak is registered at C. The distance $A'C'$ represents the time required for the pressure wave to travel from the valve to the leak and back to the valve. If the velocity of transmission of the pressure wave is known, the distance from the valve to the leak is easily computed. The difficulty in the use of this method is in the determination of the velocity of transmission Z of the pressure wave. The velocity of the pressure wave will vary somewhat according to the amount of air in the water. Another method which avoids the necessity of determining the value of Z is as follows: When the indicator pencil reaches E the first relief of pressure due to the source is felt. The distance $A'E'$, therefore, represents the time required for the pressure wave to travel from the valve to the source and back to the valve. If the length of the pipe line from the valve to the source is L, the distance from the valve to the leak can be determined by proportion:

$$l : L = A'C' : A'E'$$

In the writer's experiments much more consistent results were obtained by this method than by the use of the velocity of transmission of the pressure wave and the time required for the pressure wave to go from the valve to the leak and return as scaled from the diagram.

The following values are taken from experiments made by the writer in 1906. The last two were measured from the second diagram shown in Fig. 5.

Calculated distance, feet	Actual distance, feet
64	72
70	72
371	381
385	381
375	381
262	265
265	265
116	113

The quantity of water discharged from the leak can also be determined from the water-hammer diagram. The expression for the velocity of flow in the pipe due to the leak is:

$$w = \frac{P - P_1}{h} \Big/ \left[\left(\frac{P + P_1}{2p} - 1 \right)^{0.5} - 1 \right]$$

where P is the amount that the pressure is increased due to water hammer; P_1, the amount that the pressure at the valve is above the original pressure after the return wave from the leak reaches the valve; p, the original pressure at the valve, and h the water-hammer coefficient.

A number of values have been computed

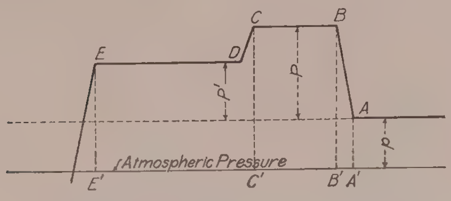


Fig. 3—Characteristic Features of Diagram

from the equation, and the results have been plotted in Fig. 4. It was assumed that $P = 55$ lb. per square inch, $p = 40$ lb. per square inch, and $h = 55$. It will be seen from an examination of the curve that even small leaks will produce a noticeable fall of pressure on the water-hammer diagram.

The first suggestion that the water-hammer diagram could be used to determine the location of a leak was made by Professor Joukovsky as the result of a series of experiments made in 1897 and 1898 for the waterworks department of Moscow, Russia. He published a monograph ("Stoss in Wasserleitungsrohren") in 1900. A translation of this paper, somewhat modified, was published in the "Proceedings" of the American Waterworks Association in 1904. Experiments were made by the writer in 1906 in the hydraulic laboratory of the University of Illinois. Fig. 5 shows two diagrams taken at that time.

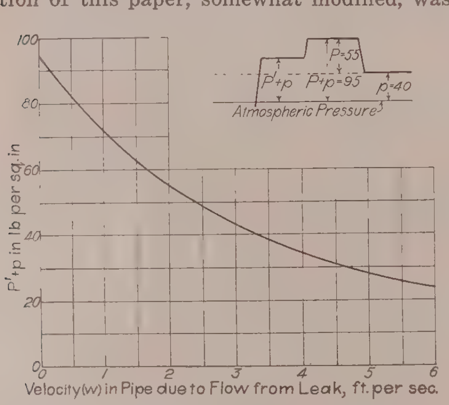
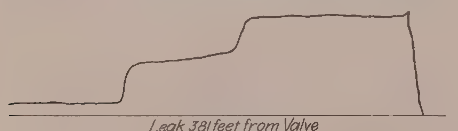
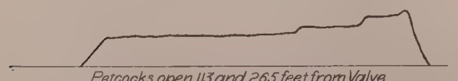


Fig. 4—Pressure Reduction from Leak

In using this method for determining the location of a leak the following suggestions are made: The quick-closing valve should be at the end of the section of pipe to be tested. This can be accomplished by tapping the main close to a valve, the valve in the main being kept closed during the experiments. The pipe leading from the main to the quick-closing valve must be large enough so that a water-hammer pressure at least as great as the static pressure can be caused by the sudden closure of the valve. The indicator should also be connected at this point of the main, or to the pipe containing the quick-closing valve. If possible, the method of proportional distances should be used. The distance to the source (large main) should be measured. A hydrant partly open will make a good reference point in case the main on which the experiments are being made is very long. When the method of proportional distances is used it is not necessary to know the speed of the paper. It is only necessary that the paper travel at a uniform speed while the diagram is being taken.



Leak 381 feet from Valve



Petcocks open 113 and 265 feet from Valve

Fig. 5—Diagrams Obtained by Experiments

[In the Engineering Record of Sept. 13, 1913, page 289, there is an abstract of a paper by William W. Brush, deputy chief engineer of the Department of Water, Gas and Electricity, New York City, presented before the New England Waterworks Association, describing an apparatus called the "pulsograph" and invented by N. W. Akimoff, of Philadelphia. The device is designed on the principle described by Professor Enger in the foregoing paper.—EDITOR.]

Water-Power Legislation in British Columbia

Method Designed to Encourage Legitimate Development and Discourage Speculation

THE British Columbia Water Act of 1909 has lately been amplified by a set of "rules and regulations" which more completely specify and explain the legal procedures necessary in acquiring and developing Government water powers, and also establish a new system for the collection of revenue from the users of such powers. These new regulations have been drawn up after an extensive study of British Columbia conditions in the light of the most approved methods in use in the United States, and are designed, primarily, to encourage legitimate development and to discourage speculative promotion.

Since passing the 1909 act, which was modeled after similar legislation in Wyoming, rapid strides in the management of water rights have been made in British Columbia, and the Dominion Government recently turned over to the Province the water powers within the railway belt, so that now all streams in British Columbia are under its own jurisdiction. Dr. S. Fortier, of the U. S. Department of Agriculture, spent two months in British Columbia and submitted recommendations for the reorganization of the water rights branch, and in order to carry out Dr. Fortier's policy, it is reported, H. W. Grunsky, one of his own assistants, was retained at Victoria in an advisory capacity. The branch also organized an engineering staff and divided the work into three classes, namely, the determination of existing rights, the conditions under which new rights would be granted, and the supervision and administration of established rights. The staff attending to the first of these three subjects has found an enormous quantity of work awaiting attention, and up to the first of the present year one-third of the 7250 water records to be adjudicated had been investigated. Of these 1031 were confirmed and 483 were canceled.

The new rules and regulations cover matters which were only partially provided for in the water act of 1909 and go into full detail in regard to (1) petitions and fees, (2) expropriation by municipalities, (3) the use and occupation of Crown lands, (4) fees for the use of water for domestic, irrigation and other purposes, (5) fees for the use of water in the development of water power, (6) head gates, structures and measurement of water, and (7) the filing of plans, office procedure and fees.

Under these several headings a new policy has been laid down which will make a radical change in the administration of provincial water rights. The comptroller, whose duty it is, as chief of the water rights branch, to guard the granting of new rights, as well as to supervise matters of an engineering and administrative nature, is given considerable judicial authority in settling questions and disputes; while the executive council can order a license revoked in certain cases, such as a refusal or failure to develop the full amount of power covered by the license within a period of five years after there is a public demand for such power. The outstanding feature of the new policy, however, is in the manner of determining the fees which will, from now on, be based on the capacity of the site as measured by the net horsepower which can be produced at 70 per cent efficiency. This basis for determining the fees is thought preferable to a charge on the actual horsepower output, because it virtually allows the licensee a premium on highly efficient machinery at the same time that it limits the dealing of the Government purely to the water-power side of the matter. Another purpose which this basis of rating is to accomplish is the discouragement of small developments on large power sites by means of which a company might secure and hold undeveloped an important privilege if the fees charged were based only on the output of the plant and not on the potential power of the site.

BOND REQUIRED

IN order to exclude at the outset all who are not bona-fide investors, a substantial sum is required as a record or application fee. A bond is also required, which is designed to relieve the licensee from high rental charges during construction and early operation periods, at the same time that it gives the Government a guarantee of good faith. The bond varies from 50 cents to \$2.50 per horsepower as the total power of the site decreases from 100,000 to 1000 hp., and is not to be less than five times the amount of the record fee. All rental payments made during the survey-construction period are rebated to the licensee as credit upon the rental account in the operation period, this being designed to lessen the expenses of early operating periods, as well as to hasten the completion of the plant. The amount of the rental fee during the operating period, which fee is fixed by the board of investigation, is in no case to be less than 25 cents nor more than \$1 per horsepower per annum.

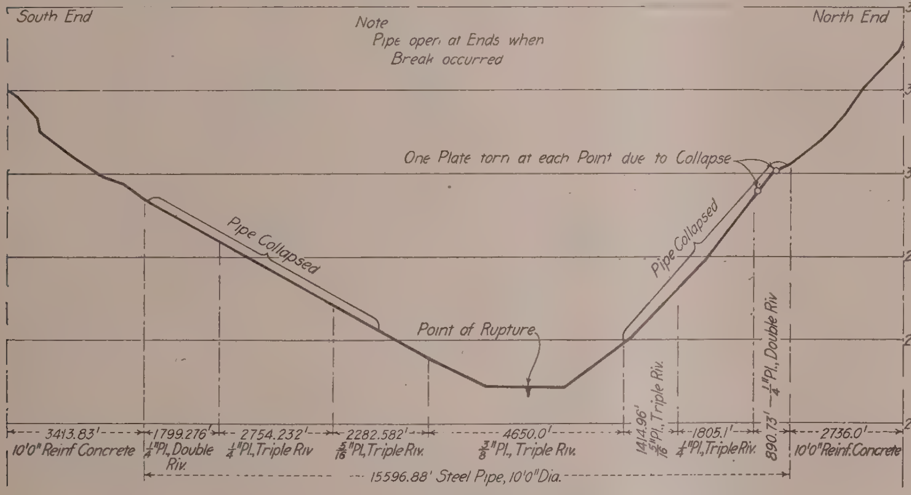
INVESTIGATIONS OF LIGNITE AND PEAT carried on by the Bureau of Mines during the year ended June 30, 1913, according to the report of the director, include field studies of the nature and origin of peat to determine how peat deposits might best be used, chemical analyses to determine the fuel value of the beds examined as compared with the usual fuel in use in the same region, microscopic and physical analyses to determine the best ways to make raw peat ready to use and structural study to determine the relation of different types of peat to each other and to lignite and coal. The report points out that while between 15,000,000 and 20,000,000 tons of peat are annually produced and used in the countries of Europe, yet only 1300 tons were reported produced and used in 1912 in the United States, where it is burned by the owners merely to get rid of it.

Floods Damage Antelope Siphon, Los Angeles Aqueduct

When Piers Were Undermined Pipe Ruptured and Flattened Out as Water Escaped

By WILLIAM MULHOLLAND
Chief Engineer, Bureau of Waterworks

IN the general havoc spread by the great storm which visited Southern California during the last week of February the Los Angeles Aqueduct did not escape unscathed. On the stretch of 38 miles of open, concrete-lined conduit extending from the Alabama Hills to the Haiwee reservoir some sand washed into the conduit, but the amount of the damage in this instance was almost negligible and the flow of the aqueduct was not materially interfered with. The only other point at which trouble was experienced was the Antelope siphon, a structure of steel and concrete by which the waters of the aqueduct are carried across the western end of the Antelope Valley at a distance of about 100 miles from Los Angeles. The aqueduct was under operation at the time, and with the rupture of the steel at two different points the water rushed out, causing the pipe to collapse.



Extent of Damage to Steel Pipe of Antelope Siphon, Los Angeles Aqueduct

The storm in the vicinity was of most unusual proportions. The average annual rainfall in this vicinity is between 7 and 9 in. The rainfall of the present season, however, has so far amounted to 34.25 in. This record has been increased by a continuous rain on Feb. 18, 19 and 20, amounting for the three days to 11.6 in.; the observations were taken from the C. P. Collins gage, about 4 miles south of the south end of the Antelope siphon and about 5 miles from where the rupture causing the damage took place. The gage shows that for the 24 hours of Feb. 20 the rainfall amounted to 6.8 in.

Collapsed Pipe Section

The Antelope siphon has a total length of 21,746.71 ft. At the north end there are 2736 ft. of 10-ft. reinforced concrete pipe; 890.73 ft. of ¼-in. plate, double riveted; 1805.1 ft. of ¼-in. plate, triple riveted; 1414.96 ft. of 5/16-in. plate, triple riveted; 4650 ft. of ⅜-in. plate, triple riveted; 2282.582 ft. of 5/16-in. plate, triple riveted; 2754.232 ft. of ¼-in. plate, triple riveted; 1799.276 ft. of ¼-in. plate, double riveted. All of the steel pipe is 10 ft. in diameter. There are 3413.83 ft. of 10-ft. reinforced-concrete pipe, comprising the south end of the siphon. The maximum head on the concrete pipe is 75 ft. and on the steel pipe 200 ft.

When the siphon was constructed, during the year 1912, the small ravine, approximately 15 ft. wide and 8 ft. deep, which carried the drainage at the west end of the valley was spanned by the siphon supported on two concrete piers, one on each side of the wash. The siphon for the remaining portion of its length lies half buried in the ground. This ravine gave no indication of ever having carried a flow of more than 40 or 50 sec.-ft. Some time during the night of Feb. 20 the heavy rainfall of the preceding 24 hours increased the already heavy flow through the ravine to one of flood proportions that cut a channel over 20 ft. in depth and nearly 150 ft. in width.

The two piers were undermined and fell over on their sides intact. This allowed the pipe to drop and three of the round seams were torn asunder. With the emptying of the siphon the ¼ and 5/16-in. pipe collapsed. The heavy plate remained uninjured, as did also the reinforced-concrete pipe.

On Feb. 23 the order for the plates necessary to repair the rupture was given to a local shop and on Feb. 25 a gang of riveters was dispatched to the scene. The plates were shipped on March 1 but went astray on the railroad and were not delivered at Mojave until 8 days after shipment, causing that much delay in the repair work that followed. In addition to the breaking down of the siphon at the ravine there were two points near the north end where the plates were ruptured due to the collapse.

Pipe Fully Restored

When the pipe was joined together at the lowest point, the water was immediately turned in and the collapsed portion rounded out perfectly. This was on March 19.

As shown by the profile, there were no summits in the line, the ends being wide open to the atmosphere. It will hence be seen that the absence of air valves in no manner accentuated the conditions. From a money point of view, the mishap has proved of minor importance.

Joint Commission's Remedies for Boundary Water Pollution

REMEDIES for the pollution of the boundary waters between the United States and Canada were discussed at a meeting of the International Joint Commission, held in Washington, April 7. The first portion of the commission's work was completed recently when its report on the location, origin and extent of pollution was submitted (see Engineering Record of Jan. 31, page 123). The task now is to determine by what methods it is possible or advisable to prevent this pollution.

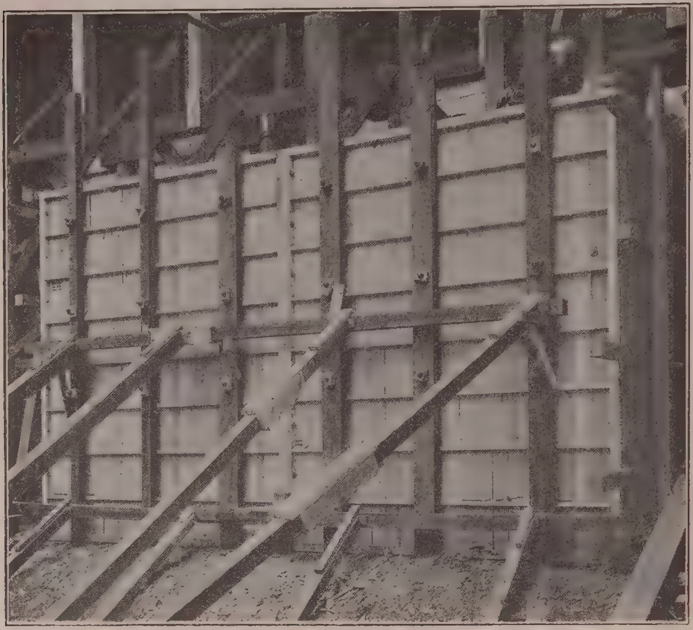
The plan suggested by Dr. Allan J. McLaughlin at the meeting was substantially as follows:

First, secure the opinions of the most eminent sanitary engineers by submitting to them a list of questions. These opinions would form a sound basis for formulating a general policy and possibly some general minimum requirement.

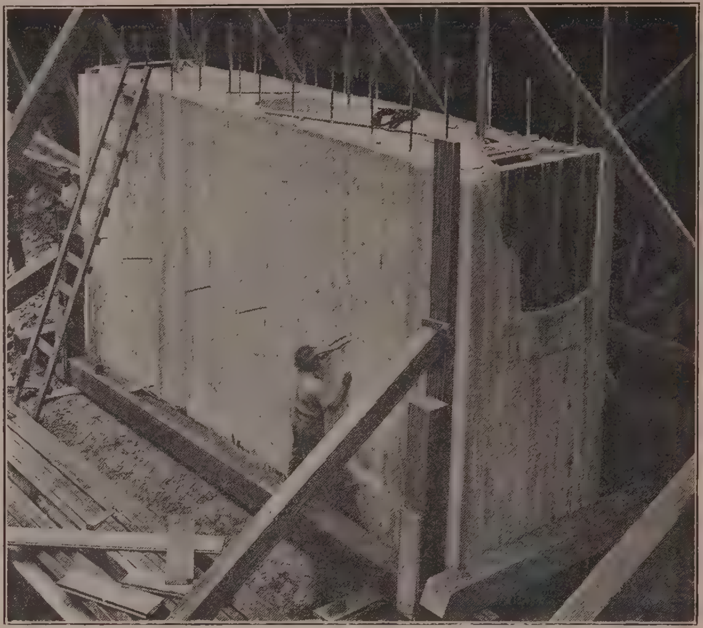
Second, hold public hearings at which the municipalities could present their cases, including expert engineering testimony, plans and projects.

Third, employ expert sanitary engineers for the formulation of special standards for each waterway in excess of general minimum requirements depending upon local conditions and necessities.

It is probable that the commission will proceed somewhat according to this outline. In so doing and in carrying out the third step—formulation of special standards—it will realize the second of the essentials which is considered necessary for the protection of the public health—viz., a standard for the raw water at the intake. The standard for raw water at the intake, the commission believes, is the only means by which an equitable adjustment can be secured between the two agencies in producing safe water, water purification and sewage treatment.



Working Chamber Forms Ready to Concrete



Finished Section of Piers and Working Chamber

Equitable Building Foundations

Concrete Piers with Footings 18 Feet below Cutting Edges of Pneumatic Caissons Sunk through 45 Feet of Sand and Water

CONCRETE piers for the columns of the Equitable Building, New York, were sunk through about 45 ft. of sand and water by the pneumatic caisson process, and their foundations were carried 16 to 20 ft. deeper to solid rock by excavations made below the cutting edges through a stratum of hardpan not penetrated by the caissons. They were sealed in the upper part of the hardpan and the latter was found to be sufficiently impervious, hard, and tough to retain the air pressure and permit excavation without sheeting or other retaining. On this work the excavation was extended below the cutting edge much farther than is usual under similar circumstances, an arrangement which expedited the work and reduced its cost.

All four sides of the 312 x 167-ft. lot were enclosed by rectangular piers extending from about 10 ft. above ground-water level to the surface of the rock and serving both to support the wall columns, retain the exterior soil, and exclude the ground water from the sub-basement which was carried down 15 ft. below water line. At each corner of the lot there was a 12 x 12-ft. or 12 x 13-ft. caisson, intermediate between which there were thirty-three rectangular caissons of a uniform width of 6 ft. and of lengths varying from 16 ft. 8 in. to 51 ft. 1 in., most of them being 25 ft. long. They were sunk exactly in line on each side of the lot, with a nominal clearance of 3 in. between adjacent ends, and after sinking were concreted together, making a continuous wall or dam enclosing the lot, as has been described in the Engineering Record for various other buildings in New York.

The sinking of these caissons in advance of other foundation work enabled the lot to be drained so that the eighty interior piers could be built in open excavation in cofferdams or sheeted pits, thus effecting economy and convenience by the elimination of a large amount of pneumatic caisson work which would otherwise have been required.

The caisson working chambers, 6 ft. high, were built of monolithic concrete continuous with the pier shafts, and were constructed wholly above the surface of the ground in removable wooden forms of the standard type generally used by this contractor. They were erected on the bottom of the preliminary general excavation at an elevation of 13 ft. above ground-water level and about 21 ft. below the curb. Vertical bents, about 6 ft. high, made of 3-in. planks, were braced together on the level surface of the ground in exactly the required positions. They were covered with lagging for the inner surface of the working chamber walls, battered and offset as required, and capped with horizontal lagging to make the bottom of the form.

CAISSON WORK

THE 6 x 6-in. steel angle-cutting edge was secured in position, the vertical and inclined reinforcement rods were wired to horizontal pieces, and the first 16-ft. course of the outside form was set up enclosing them. Temporary collapsible sectional steel air shafts in sections 6 ft. long were set up over holes through the deck, and the first course of concrete was deposited around them. This was only carried up to a height of about 2 ft. above the top of the working chambers and allowed to set for about 48 hours, thus forming a concrete slab strong enough to carry the weight of the upper portion of the pier shaft from the solid walls of the working chamber without imposing an excessive load on the centering.

The collapsible outside form was made of large panels, usually six in number, having 2-in. dressed vertical planks nailed to horizontal wales about 30 in. apart vertically. Vertical stiffening pieces at the ends of the wales formed flanges through which the panels were bolted together. The forms were braced by three tiers of horizontal transverse rods ¾ in. in diameter with nuts bearing on heavy vertical timbers engaging the waling pieces.

After the concrete had set at least 24 hours, the outside forms were unbolted, the tie-rods disconnected from the vertical strips, and the panels elevated about 15 ft. and reassembled with the lower part engaging the top of the concrete. A new course was concreted and so on, until the pier was built to the full required height. The projecting ends of the transverse tie-rods were cut off and the rods remained permanently embedded in the piers.

The caisson sinking was carried on continuously under a maximum pressure of about 22 lb. which permitted 8-hour shifts. Caisson 53, supporting two columns on the Nassau Street front of the building, was sunk through 45 ft. of sand to a penetration of about 6 in. in the hardpan where it remained stationary, and the cutting edge was thoroughly sealed with clay plastered all around in the usual manner. Most of the other caissons were also puddled with clay, but in a few instances concrete was used instead.

Below the cutting edge the excavation for Caisson 53 was carried down 18 ft. through the hardpan to the surface of the rock which was here hard and sound and approximately level. The hardpan was excavated in benches by the use of Ingersoll-Rand and McKiernan-Terry pneumatic hand drills making transverse rows of 1¼-in. vertical holes about 12 in. apart, and 3 ft. deep, spaced about 2 ft. back from the face of the bench. These enabled the hardpan to be broken off without the use of any explosives, after which it was sledged if necessary into pieces that were hoisted in steel buckets through the two O'Rourke air locks.

CONCRETING

THE caissons involved about 12,000 yd. of excavation and the same amount of concrete and the interior piers involved about 8000 yd. of excavation and of concreting. The 1:2½:5 concrete was made with Pennsylvania Portland cement mixed in Ransome and Smith machines, hoisted in movable towers, and delivered to the forms through Lakewood system chutes, as described in the Engineering Record of Nov. 15, 1913, page 550.

The working chambers and air shafts were filled through concrete locks that enabled the concrete to be deposited much more rapidly than could have been done by buckets passing through the regular material locks. These locks consisted of steel hoppers of about 1-yd. capacity, set in the holes provided for the upper doors of the regular air-locks. The lower door of the latter was kept permanently open and the caisson air pressure here was retained by the air-tight top and bottom doors in the hopper. The lower door covered the entire area of the bottom of the hopper and was commanded by a chain controlled from the top of the hopper, outside. Both doors opened downward in the usual manner and were provided with gaskets.

While the hopper was empty and its upper door was closed, the bottom door was hauled up into closed position by its chain and the air-pressure being exhausted from the hopper permitted the upper door to be opened while the lower door was held firmly in position by the unbalanced caisson pressure below it. The hopper was then filled with concrete, the upper door closed, and air-pressure admitted to the hopper. This pressure, together with the weight of the concrete, quickly overcame the pressure on the under surface of the lower door and caused the latter to open and automatically discharged the concrete through the air shaft. The lower door was then pulled up from the outside, the operations repeated, and so on. A maximum of fifteen hoppers per hour were thus delivered into the caisson.

The cellar excavation extended 50 ft. below the curb and involved extensive timber bracing, reaching from Broadway to Nassau Street and from Pine to Cedar Street. The side pressures were very high, owing to the unstable character of the ground and the superimposed weights of several high buildings whose foundations were only level with the tops of the cofferdam caissons. The work was done without damage to adjacent skyscrapers, although the failure of any of the braces might have ended in a disastrous collapse.

D. H. Burnham & Company are the architects, and Daniel E. Moran is consulting engineer. The Thompson-Starrett Company is the general contractor, and the O'Rourke Engineering Construction Company is the engineer and contractor for the foundations, excavations and shoring.

ELECTRIFICATION OF THE 30-MILE railway between Savona and Ceva, Italy, has been started, and the 27-mile line between Savona and Genoa is to be electrified within three years, according to a consular report.

Chicago Engineers' Rule Book

Board of Public Works' Suggestions to Employees in Dealings with the Public

SAFETY first of all, then service, and third, efficiency are required of employees of the Chicago Bureau of Engineering in the Board of Public Works, which has recently issued rule books to the various divisions. The book is a 29-page, 4 x 6-in. pamphlet, in which rules are laid down for the guidance of employees in their dealings with the public and with one another. It opens with a letter by John Ericson, city engineer, who states that in dealing with the public the first endeavor must be to render the service to which it is entitled. Prompt and courteous attention to all reasonable requests and complaints will give the public a right opinion of the bureau. Remember, he says, in all your dealings to be guided by the rule, "Do unto others as you would have them do unto you."

Of equal importance with courtesy and consideration is the matter of safety. The employees of this bureau are entrusted with the guarding of the safety of the citizens, particularly in the matter of water supply and of traffic across bridges, where negligence or carelessness would jeopardize the lives and health of the community. Take no chance which will risk the lives or safety of the public, allow nothing to interrupt the service to which they are entitled, and see that all city business is handled efficiently and economically.

PERSONAL INTEREST REQUIRED

EVERY employee of the bureau should take a personal interest in all the work of the bureau and should consider himself on duty at all times, both during regular working hours and outside of them. He should report promptly anything which occurs to his attention as needing repair, such as a water leak in the street or in any building, a broken sidewalk or roadway on a bridge, or any other matter coming under the jurisdiction of this bureau. If all employees take such an interest in their work they will find that they are able to take pride in belonging to the Bureau of Engineering.

As the city engineer has not time to examine in detail all plans and specifications submitted for his approval and at the same time a careful check must be made, therefore, everything must first be presented to the assistant city engineer for examination and aproval. Preliminary discussions are to be held to avoid changes, and the argument that a drawing has been traced or a specification has been printed will not amount to anything. If a change is necessary it will be made in order to make the specifications and plans satisfactory to the city engineer.

Therefore, it will be wise to go over such plans and specifications before they are finally printed or traced rather than afterward.

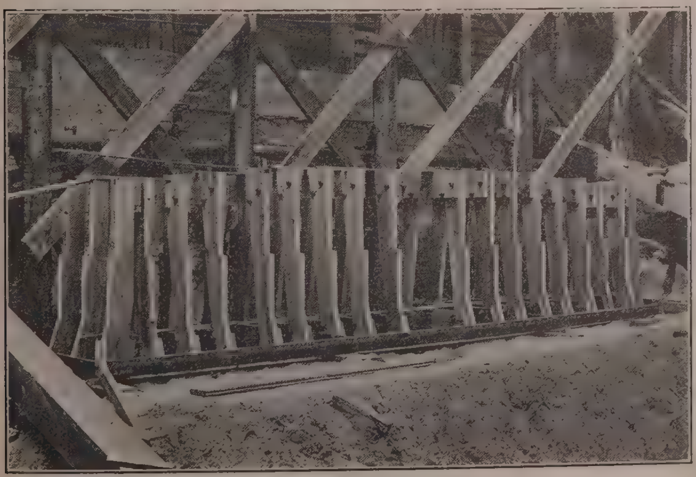
Details are given of making out bidding tabulations and letters of recommendation conforming with the necessary legal routine. It usually requires thirty days' time for the law department to approve, and the comptroller and Mayor to sign, a contract.

PROGRESS SCHEDULES

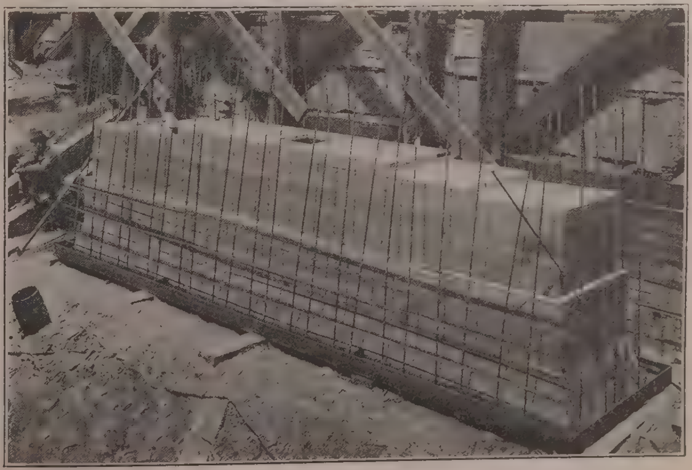
CAREFUL attention to rate of construction is noted and engineers in charge of work are admonished to make up schedules of progress and notify the contractor when he falls behind. Call in the police is the last resort, following engineers' orders and notices, when contractors fail to stop work, not complying with specifications. Any engineer who passes estimates while at the same time he is protesting work will be charged with inefficiency before the Civil Service Commission. Lack of foresight and care shall not be an excuse for emergency and rush orders rather than the regular requisition method of obtaining material.

Street car tickets issued, exclusively for city business, do not include going to and from homes. Dismissal is the penalty for non-observance of this rule. Duplication of correspondence requiring the city engineer's signature is avoided by engineers directing letters to the addressee and signing in lower left-hand corner. Carbons for as many men as must sign on different colored paper must accompany the original. No employees shall have any other employment or any association with outside interests under penalty of dismissal.

Special chapters deal with the duties of the employees and particular rules for each division. For instance, in the water meter shop it is stated that there is no limit to the amount of work an employee must do. A good day's work for a good day's pay is what is expected from each and every one. Employees shall never pass up a job when it is possible to do it. If a little digging is required or meter is obstructed and obstruction can be removed, the obstruction shall be removed and reported; each meter setter has a laborer with him; that is part of his duties. All employees shall be courteous to the publc with whom they come in contact. They shall explain but not argue.



Framework for Inside Form for Working Chamber



Inside Form, Reinforcement and Cutting Edge for Chamber

Comparison of Systems of Flood Control

Third Paper in This Series Dealing with the Storage of the Flood Producing Runoff Even When No Advantageous Reservoir Sites Exist

By H. A. PETTERSON
Civil and Hydraulic Engineer, San Francisco

THIS paper will analyze the elements entering into the problem of what percentage of the area of a watershed it is necessary to control by reservoirs in order to prevent floods—in other words, what percentage of the maximum discharge is it necessary to control? The Sacramento watershed will be referred to in this discussion, not because it offers favorable conditions for regulation by reservoirs, but because it presents great difficulties. The large disparities in mean and maximum flows of the streams of this watershed are shown in Table 1. Because of the great value of water for irrigation and power development it is essential that the runoff from this watershed be regulated.

Two important factors in the prevention of floods are maximum rate of runoff and capacity of the stream's channel, for the cause of floods is simply the inability of the stream to carry all of the waters pouring into it from the tributary watershed.

VARIOUS CASES

FOUR cases are considered in this discussion, differing in the assumptions of the rate of outflow allowable from the reservoir.

Symbols: F = the maximum twenty-four-hour discharge in second-feet, from the watershed considered; Q, the average normal discharge in second-feet from the watershed under consideration; c, the ratio of channel capacity to normal discharge; cQ, channel capacity in second-feet; r, percentage of runoff F that must be controlled by storage, and M, the mean discharge in second-feet of the minimum year from the watershed under consideration.

Case 1: The conditions are that the total runoff tributary to the storage reservoir is impounded during the flood stages and none is let out during high water. Hence the runoff from the uncontrolled portion of the watershed shall not exceed the channel capacity. (This case is possible if the reservoir is to be used for irrigation and navigation but not for power development.)

rF = runoff impounded in reservoirs; $F - rF$ = the runoff from the uncontrolled watershed = cQ

or
$$r = (F - cQ)/F \qquad (1)$$

Case 2: The conditions are that the runoff from the unregulated portion of the watershed plus a reservoir outflow M shall not exceed the channel capacity. (This rate of outflow must be maintained at all times for power development, water supply or other uses.)

rF = runoff flowing into the reservoir; M = the outflow from reservoir; $rF - M$ = the discharge being stored up; $F - rF + M$ = the total runoff contributed to the stream below = cQ, or
$$r = (F + M - cQ)/F \qquad (2)$$

Case 3: The conditions are that the runoff from the unregulated portion of the watershed plus a reservoir outflow Q shall not exceed the safe channel capacity.

rF = the runoff into the reservoirs; Q = the outflow from the reservoirs; $rF - Q$ = that portion of the runoff that is being impounded; $F - rF + Q$ = total stream flow below the reservoirs, = cQ, or
$$r = (F + Q - cQ)/F \qquad (3)$$

Case 4: The conditions are that the runoff from the unregulated portion of the watershed plus a reservoir outflow cQ shall equal cQ. It is self-evident that for this case the entire watershed must be regulater. Or
$$r = 1 \qquad (4)$$

Values of r for cases 1, 2 and 3 respectively are given in Table 2 for a number of river basins. Formulæ 1 to 3 seem so simple as to be nearly self-evident, yet the writer has never seen anything like them in print. On the other hand, he has read a great many statements that indicated a lack of understanding of the simple truths contained in these formulæ.

These formulæ explain why the reservoirs on the Upper Mississippi, for instance, which control only a small percentage of the catchment area, fail to prevent floods, or fail to maintain a navigable depth from St. Paul to St. Louis in years of drought.

MEANING OF r

IT must be borne in mind that r is not the percentage of the drainage area but of the flood runoff. There are always parts of a watershed whose runoff does not materially increase flood height, because its waters reach the trunk stream long before it has become swollen. There are other areas, so distant in point of time from the flood-producing parts of the watershed, that their contributions do not arrive until after the subsidence of high water in the main stream. The Pittsburgh Flood Commission brings this out very clearly in its flood studies on the Allegheny and Monongahela.

The value of r for the Upper Mississippi is given in Table 2 as 67.3 per cent for Case 2. That is, when the flood discharge is 450,000 sec.-ft., the storage reservoirs must be so situated as to be able to reduce 67.3 per cent of this discharge to an outflow of 53,500 sec.-ft. (the mean flow of the minimum year) and the remaining discharge is uncontrolled. The total stream flow below the reservoir is made up as follows:

Outflow from reservoir............	53,500 sec.-ft.
From unregulated water shed, 32.7 per cent. of 450,000..............	146,500 sec.-ft.
Total = cQ = ...	200,000 sec.-ft.

Now conditions might easily exist (and undoubtedly do on many watersheds) where two-thirds of the flood flow is produced from, say, one-half of the watershed. If the reservoirs are located so that the catchment areas with high flood yield are all tributary to them, the percentage of drainage area necessary to be controlled will be far less than the values of r given in the table. Thus the percentage of the drainage area for the Upper Mississippi might easily be 50 per cent of the total watershed in place of the 67.3 per cent, which is the value of r as given in the table.

It is stated in the footnote to Table 2 that for tributaries of the Sacramento, and also for the Mississippi, the values of c were so chosen that the summation of the products of cQ would equal 120,000 sec.-ft. and 1,000,000 sec.-ft. respectively. Perhaps this statement should be amplified, using the Mississippi as an example. The capacity of the river at Cairo at bankful stage is 1,000,000 sec.-ft. The sum of the normal discharges of the Missouri, Upper Mississippi and Ohio, which make up the flow of the Mississippi at Cairo, equals 418,000 sec.-ft., 1,000,000/418,000 = 2.39, the value of c to be used for each stream.

Different values of c can be assigned to each tributary, more in accord with the channel capacity prevailing in these rivers—the limiting conditions being that the summations thereof, the products of cQ shall be 1,000,000 sec.-ft. Some streams might be completely regulated to uniform flow, while the runoff from other catchment areas with low flood flow and poor reservoir facilities might be entirely unregulated. It is evident, when we begin to consider the tributaries of the tributaries that a great number of combinations might be made. The best results can only be reached after careful study.

TABLE 1.—RUNOFF DATA FOR WATERSHEDS OF SACRAMENTO RIVER

Stream	Gaging Station	Water-shed sq. miles	Years of gaging records	Mean runoff in.	Mean runoff sec.-ft.	Maximum 24 hr. daily flow, sec.-ft.	Ratio of maximum to mean flow	Average of min. year's flow, (b) sec.-ft.	Ratio min. to mean. per.cent.
Sacramento	Red Bluff	10,400	9	20.6	15,800	254,000	16.2	10,720	67.7
Feather	Oroville	3,640	9	32.7	9,025	187,000	20.7	4,840	53.6
Yuba	Smartsville	1,220	8	50.7	4,550	111,000	24.4	2,203	48.3
Bear	Van Trent	263	7	31.4	610	25,800	42.3	260	42.7
American	Fair Oak	1,910	7	40.6	5,710	105,000	18.4	2,010	35.2
Stony Cr.	Fruto	601	10	18.3	810	29,300	35.9	486	60.1
Cache Cr.	Yolo	1,230	9	10.6	965	20,100	20.8	346	36.0
Putah Cr.	Winters	805	5	11.9	710	30,000	42.2	275	39.0
Totals		20,069		25.9	38,180	762,200 (a)	14.0 (a)	21,140	55.3
Unmetered area		3,907				76,000			
Valley		4,250				25,500			
Grand total		28,226				863,700 (a)			

NOTE—(a) Not all these tributaries are in maximum flood simultaneously. The maximum 4 day runoff during the flood of March, 1907, was estimated at 530,000 sec. ft. A maximum of 600,000 sec. ft. will be provided for in present reclamation plans.

(b) Minimum yearly flow is the discharge available for irrigation and power development.

CREATING ARTIFICIAL RESERVOIRS

TABLE 2 shows that for the Sacramento Basin it is necessary to control from 80 to 90 per cent of the flood-producing runoff. To do this, reservoirs would have to be constructed in the foothills, between the steeper mountainous watersheds and the valleys. Storage basins so situated would be sufficiently elevated to supply water by gravity for irrigation in the valleys.

At first thought every engineer would state that the creation of such reservoirs

TABLE 2.—PERCENTAGE OF FLOOD RUNOFF THAT MUST BE CONTROLLED BY STORAGE

River basin	Discharge: Maximum F sec.-ft.	Discharge: Mean Q sec.-ft.	Discharge: Minimum M sec.-ft.	Channel capacity cQ sec.-ft.	Value of c	Value of r: Case 1 (F-cQ)/F per cent.	Value of r: Case 2 (F+M-cQ)/F per cent.	Value of r: Case 3 (F+M-cQ)/F per cent.
Upper Sacramento	254,000	15,800	*10,720	†49,650	3.14	80.4	84.7	86.8
Feather	187,000	9,025	4,840	†28,350	3.14	85.0	87.4	89.2
Yuba	111,000	4,550	2,200	†14,300	3.14	87.0	88.9	91.0
Bear	25,800	610	260	†1,915	3.14	92.5	93.3	94.6
American	105,000	5,710	2,010	†17,900	3.14	83.0	84.9	88.3
Stony Creek	29,300	810	486	†2,540	3.14	91.1	92.9	94.0
Cache Creek	20,100	965	346	†3,030	3.14	85.0	86.8	89.7
Putah Creek	30,000	710	275	‡2,230	3.14	92.6	93.6	94.9
San Joaquin	168,500	11,740	*5,870	‡23,480	2.0	86.0	89.5	93.0
Kern	9,505	1,515	*757	‡3,030	2.0	68.0	76.0	84.0
Tulare Lake Basin	39,290	4,050	2,025	‡8,100	2.0	79.3	84.2	89.7
Missouri	900,000	113,000	56,500	§200,000	1.77	77.9	84.0	90.3
Upper Mississippi	450,000	107,000	53,500	§200,000	1.87	55.6	67.3	79.3
Ohio	1,400,000	198,000	99,000	§600,000	3.03	57.1	64.0	71.1
Arkansas	450,000	63,750	31,875	‡127,500	2.0	71.7	78.7	85.9
Red	220,000	57,100	28,550	‡114,200	2.0	48.0	61.2	74.5
Colorado	149,500	23,100	11,550	45,000	2.0	70.0	77.6	85.2
Nile (Egypt)	459,000	128,000	64,000	‡256,000	2.0	44.3	58.1	72.1

*Minimum year's runoff is uniformly taken at one-half the normal, except for tributaries of the Sacramento.

†Value of c is given tributaries of the Sacramento so that summation of the products of cQ will equal 120,000 sec.-ft.

‡Values of cQ for these streams arbitrarily selected as twice the normal flow.

§Values of cQ for these three tributaries of the Mississippi, arbitrarily taken so that the summation of the products of cQ will equal 1,000,000 sec.-ft., the capacity of the Mississippi at Cairo.

was entirely impracticable. The apparent lack of natural storage basins would make their cost prohibitive. This is undoubtedly true for most parts of the foothill region of the Sacramento Basin, mainly because of the rolling topography and the relatively steep slope toward the floor of the valley. In Table 3 are shown the unit volume of earthwork in embankments which entirely circumscribe the reservoirs they created. The basins so formed would be entirely artificial and owe nothing to the favors of topography, except that the ground was level. When the ground has a decided slope, there is an increase in the unit volume of the dam.

TABLE 3.—VOLUME OF EARTHWORK IN EMBANKMENTS FOR THEORETICAL RESERVOIRS ON A LEVEL PLAIN, WHERE THE STORAGE BASIN IS CREATED BY THE CONSTRUCTION OF EMBANKMENTS, COMPLETELY ENCIRCLING THE RESERVOIR

Reservoir capacity, acre-ft.	Cubic yards in dam per acre-foot of storage capacity. Water depth: 15 ft.	30 ft.	45 ft.	60 ft.
150,000	28.8	68.0	117	165.6
300,000	20.4	48	82.9	117
450,000	16.6	39.1	67.5	95.4
600,000	14.4	34	58.5	82.8
750,000	12.8	30.3	52.1	73.8
1,500,000	9.1	21.4	37.0	52.3
3,000,000	6.5	15.2	26.2	37.0
4,500,000	5.3	12.4	21.4	30.2
6,000,000	4.5	10.7	18.5	26.2
7,500,000	4.07	9.6	16.5	23.4
15,000,000	2.88	6.8	11.7	16.56
30,000,000	2.04	4.8	8.29	11.70
60,000,000	1.44	3.40	5.85	8.28

DETAILS OF DAMS IN THE ABOVE TABLE

Water depth, ft.	Height of dam, ft.	Width of crest, ft.	Side slopes	Cu. yd. per lin. ft. of dam
15	20	10	1-3	51.9
30	37	15	1-3	172.8
45	54	20	1-3	364.0
60	70	20	1-3	596.0

To show the probable cost of such reservoir construction, Table 4 was prepared. It contains costs per acre-foot of storage with the unit prices as shown in the table. It is assumed that the embankment would be placed by the hydraulic process which under favorable conditions could be done at 6 to 8 cents per cubic yard. An average price of 15 cents was used in the estimate.

A continuous line of tongue and grooved sheet piling is assumed, as a cutoff wall and concrete facing is assumed for the wet slope of the dam. The spillway would be entirely of concrete or of rock fill faced with concrete.

As to the cost of such construction, Table 4 speaks for itself. Where land is of small value, such reservoirs should seemingly be constructed for about an average of $10 per acre-foot of storage capacity. Where the ground has a decided slope as in the foothills of the Sierras, the cost will increase rapidly, being about 20 per cent greater for a slope of 2 ft. per mile, 100 per cent greater for a slope of 5 ft. per mile, and five times as great with a ground slope of 15 ft. per mile.

The writer believes that such reservoirs could be constructed in many places where land is cheap and other conditions favorable. Lest readers be frightened at the thought that the idea is new, he will refer them to the "Transactions," Am. Soc. C.E., Vol. V, 1876, which contains a most interesting discussion of levees and reservoirs.

In speaking of the tank irrigation system of Madras, Mr. Smith says:

"The extent to which it has been carried throughout all the irrigated regions of the Madras Presidency is truly extraordinary. An imperfect record of the number of tanks in fourteen districts shows them to amount to no less than 43,000 in repair and 10,000 out of repair, or 53,000 in all. It would be a moderate estimate of the length of each to fix it at half a mile. . . . The data, assumed only to give some definite idea of the extent of the system, would give close upon 30,000 miles of embankment. . . . Dry basin shaped hollows have banks carried around their ridges, and supplies were introduced from adjoining rivers by means of special canals; or long slopes, when the fall is considerable, have portions embanked, more or less regularly on three sides."

Among the most ancient of these tank reservoirs, he mentions the Ponairy in Trichinopoly, with its embankments 30 miles in length and a probable area of 60 to 80 sq. miles, now lost to the community, and the Veeranum Tank, with its 12 miles of embankments and 35 sq. miles in area, happily still in operation.

Mr. Hewson, in discussing the same article, mentions a tank constructed in the Island of Ceylon, which was made of huge stone blocks, strongly cemented together and covered with turf—a solid barrier 15 miles long, 100 ft. wide at the base, and sloping to a top width of 40 ft., extending across the lower end of a spacious valley.

TABLE 4—COST PER ACRE-FOOT OF CAPACITY OF ARTIFICIAL RESERVOIRS

Reservoir Capacity, Acre-Ft.	Embankment Cost at 15 cents per cu. yd.	Cement Lining Cost at 12 cents per sq. ft.	Wakefield Sheet Piling Cost at 7 cents per sq. ft.	Total Cost per Acre-Ft.: Including lands at $50 per Acre	Total Cost per Acre-Ft.: Including lands at $100 per Acre
Mean Depth=15 Ft.					
150,000	$4.32	$4.32	$0.52	$12.49	$15.83
300,000	3.06	3.06	0.37	9.82	13.16
450,000	2.48	2.48	0.30	8.59	11.93
750,000	1.92	1.92	0.23	7.40	10.74
1,500,000	1.36	1.36	0.16	6.21	9.55
3,000,000	0.98	0.98	0.12	5.41	8.75
6,000,000	0.67	0.67	0.08	4.75	8.09
Mean Depth=30 Ft.					
150,000	$10.20	$5.90	$0.68	$18.45	$20.11
300,000	7.20	4.16	0.48	13.51	15.17
450,000	5.86	3.40	0.39	11.32	12.98
750,000	4.54	2.63	0.30	9.14	10.80
1,500,000	3.21	1.86	0.21	6.95	8.61
3,000,000	2.28	1.32	0.15	5.42	7.08
6,000,000	1.60	0.96	0.11	4.34	6.00
Mean Depth = 45 Ft.					
150,000	$17.55	$6.96	$0.81	$26.43	$27.52
300,000	12.44	4.92	0.57	19.04	20.15
450,000	10.12	4.01	0.47	15.71	16.82
750,000	7.82	3.10	0.36	12.39	13.50
1,500,000	5.55	2.19	0.26	9.11	10.22
3,000,000	3.93	1.53	0.18	6.75	7.86
6,000,000	2.77	1.10	0.13	5.11	6.22

Note—Length of sheet piling is two-thirds the height of dam.

Tests of Grouted Concrete

TESTS of concrete specimens, of which half were hand-mixed and half made by the Hassam process—in which mortar is rolled into a previously rolled layer of broken stone—were made recently by A. W. French, professor of civil engineering at Worcester Polytechnic Institute. Two-inch stone was used in all cases and the amount of cement was the same in specimens of the two types. All were kept damp for twenty-seven days before testing.

Four beams, about 12 x 6 in. in section, of each class were tested to destruction by central loads over a 3½-ft. span. The extreme fiber stresses of rupture were as follows: Hassam beams, 574 lb. per square inch; hand-mixed, 430 lb.—indicating a superior bending strength of Hassam beams of 33 per cent. Compression tests of 18 blocks—ten of which were Hassam—averaging about 4½ x 6 x 4½ in. in size, showed the following average results: Unit stresses at first crack: Hassam block, 4845 lb. per square inch, hand-mixed block, 3910 lb.; at rupture, Hassam, 6621 lb., and hand-mixed, 4644 lb. The ultimate compressive strength of the Hassam blocks was thus 42 per cent greater than those prepared by the usual method.

Concrete Plants Used on the Rome Improvement

Numerous Small Bridges Built from Central Layout, with Interchangeable Auxiliary Equipment for Isolated Structures

IN the articles in the issues of Feb. 21, page 221, and March 7, page 282, the extent of the changes necessitated by the construction of the New York State Barge Canal was set forth and the locations of the principal concrete structures were indicated. While none of these individual structures are large, the total concrete involved is considerable, and ample opportunity was afforded to make or lose money in the design of the concrete plants. Some of the bridges, such as those for the New York, Ontario & Western and the Utica & Mohawk Valley Railways, had to be built early to make way for the rest of the improvement. The construction of other structures had to be delayed for one reason or another. All had to be co-ordinated with the grading and the operation of the railroads and the open season of the Erie Canal.

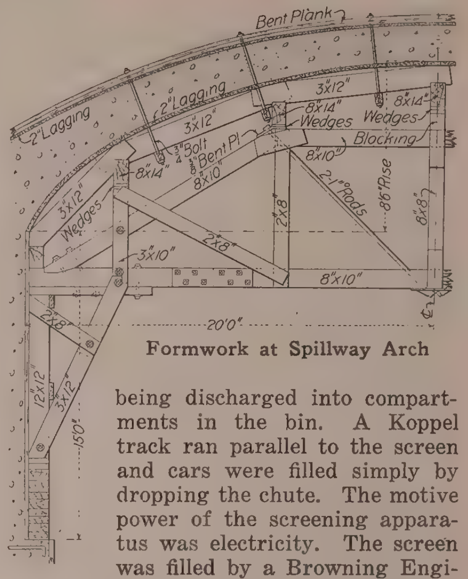
As a large percentage of the bridges are within a comparatively small area, with a few at isolated points, it was decided to build one central concrete plant to serve as many of the structures as possible, and build the others, a few at a time, with equipment that could be transferred from point to point. The structures for which concrete was mixed at the central plant included the Mill Street arches, Mill Street highway bridge, Utica & Mohawk Valley bridge, Erie Canal arch, the three New York, Ontario & Western bridges and the Lawrence Street arch, requiring a total of 14,120 cu. yd. In addition, all of the sand and gravel required for the James Street arch, about 2515 cu. yd., were prepared at the central plant, and transported to the site of the structure for use in an isolated mixer.

As stated in the previous articles, the greater part of the dredged material contains both sand and gravel that are admirable for concrete. A plan showing the layout is presented. Spoil from the dredge Stanwix was pumped to a pile, whence a bucket elevator raised it to the rotary screen above the bins. The dust was deposited in the first bin, whence it was removed by a Koppel car. The concrete ingredients were conveyed by a belt up the incline and deposited in the hopper of a Ransome mixer, from which the fresh concrete was fed into narrow-gage dump cars and hauled by dinkeys to the various structures.

The means used for handling and depositing the concrete at the different structures varied with the individual requirements at each location. On account of the material being mixed at and transported from a distant point, the handling was by derricks. The double arches were so designed that a single steel derrick with 80-ft. boom could be made to cover the structure, if located in the center of the central pier, and this method was followed at the spillway arch and the Mill Street arches. As the Erie Canal arch exceeded the reach of one derrick, the difficulty was overcome by using a movable hopper and chutes, dumping into the hopper for areas beyond the normal reach of the derrick. The Lawrence and James Street arches were covered by a single derrick, by building reinforced-concrete piers up from the foundation on the backs of the arches, which were subsequently incorporated into the concrete of the arch, upon which the foot block was set. Views are shown of the method used at Mill Street.

DOUBLE SPILLWAY ARCH

ONE of the most successful of the isolated plants was that used at the double spillway arch. This structure is adjacent to a considerable spoil bank of excellent concreting material pumped from the prism of the canal. The apparatus used for the preparation of the material consisted of an Acme Road Machinery Company bin of the usual transportation variety, upon and over which was placed a rotary screen supplied with a washing pipe, the screened material being discharged into compartments in the bin. A Koppel track ran parallel to the screen and cars were filled simply by dropping the chute. The motive power of the screening apparatus was electricity. The screen was filled by a Browning Engineering Company locomotive crane.



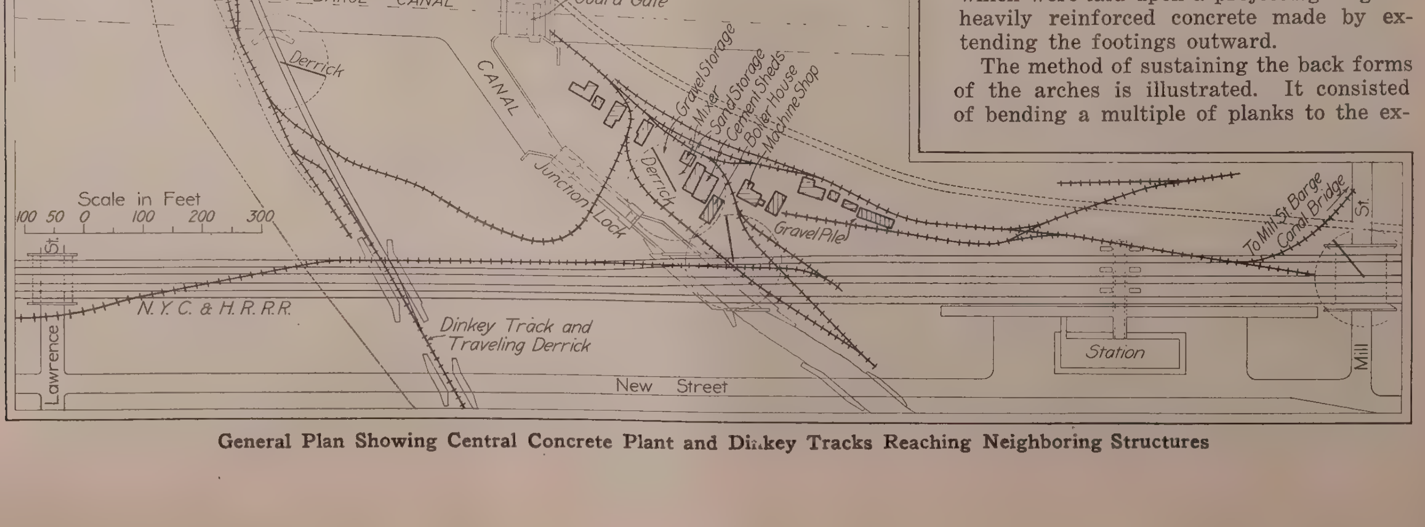
Formwork at Spillway Arch

It will be seen that the transportable screen with its loading apparatus and the means for discharging and transporting the material, were very flexible, allowing the entire screening outfit to move along the spoil bank with equal economy of operation whether near the mixing apparatus or further removed therefrom. In fact this comparatively small outfit was found to fulfil its purpose so fully and economically that it was used for the preparation of the material required for the crossing of the industrial tracks at the east end, as also for portions of the materials required for certain Barge Canal structures. The separated materials were conveyed in Koppel cars and were dumped in hoppers above a Ransome mixer. The mixer emptied into one of two cars running on two short tracks which joined at the mixer and were reached at the other ends by a derrick placed on a concrete pedestal in the center of the center pier, which reached every part of the double arch. This concrete pedestal was permanent and was made by building part of the center pier up above the arch ring in advance of the construction of the arches.

CENTERING

AS the spillway arches crossed a channel way through which most of the flow of the Mohawk River was discharged, it was impossible to support the centering by posts or piling as was the method followed in the other arch structures. A trussed center was therefore used, details of which are shown. In order to guard against unequal settlement, these trusses were supported on posts resting on sills which were laid upon a projecting ledge of heavily reinforced concrete made by extending the footings outward.

The method of sustaining the back forms of the arches is illustrated. It consisted of bending a multiple of planks to the ex-



General Plan Showing Central Concrete Plant and Dinkey Tracks Reaching Neighboring Structures terior radius of the arches and tying them in position by through bolts passing through the centering, the boarding of the form passing beneath and being held in place by the bent outer strips.

The method followed in most of the isolated plants was similar, varying only to meet slightly varying conditions, and, in the main, consisted of a Haines mixer fed by a small stiff-leg derrick. The mixer discharged into buckets on cars running on shuttle tracks and was handled to place by derricks with 75 or 80-ft. booms, controllable bottom-dump buckets being used.

The simplest layout was that used for the foundations of the new station, built after the embankment alongside was placed. Hand screens were set up on top of the embankment, the screened material from the fill being wheeled in barrows down a slight grade to a small rotary mixer placed at about the central part of the station, and the concrete wheeled to the point of deposit, the whole operation being down grade.

The M. A. Talbott Company, of Baltimore, of which Herbert M. Knight is engineer, was the contractor for all work except at the passenger station, carrying out the work under the general direction of John A. Bensel, State engineer and surveyor, for the State, and George W. Kittredge, chief engineer, for the New York Central & Hudson River Railroad.

Influence of Aluminum on Rails

Conclusions Drawn by M. H. Wickhorst, of the A. R. E. A. Rail Committee, from a Series of Tests Made under His Supervision

INVESTIGATIONS have been made at the works of the Illinois Steel Company at South Chicago by M. H. Wickhorst, engineer of tests of the rail committee of the American Railway Engineering Association, as to the influence of aluminum on bessemer ingots and rails when added to the molds while pouring the steel. A full account of the tests is given in the report of the committee. Mr. Wickhorst sums up his conclusions as follows:

Ingots treated with aluminum as mold additions were of more even composition throughout the ingot than plain bessemer steel. There was less positive segregation in the interior and upper part of the ingot but the negative segregation or soft center in the interior and lower parts of the ingot was about the same. There was a softening or negative segregation in the upper part of the wall of the plain ingot, while in the aluminum-treated ingots the walls were of fairly even composition throughout the height of the ingot.

Aluminum-treated ingots had larger and deeper pipes than plain steel, but had denser steel around the pipes.

Rails of plain steel had a brittle zone in the upper part of the bar as disclosed by the drop test. In the rails of aluminum-treated steel this zone was largely eliminated.

Rails of plain steel contained their laminations close to the top end of the bar, while in aluminum-treated rails the interior laminations were found a considerable distance from the top end, varying from about 30 to 45 per cent of the weight of the ingot.

In the transverse test of the base, rails of aluminum-treated steel with 0.61 per cent carbon showed considerably greater transverse strength of the base and sag of the flange before breaking than rails of plain steel, and those with 0.45 per cent carbon showed a little greater strength and sag of flange.

Three 36-Inch Taps in a 60-Inch Steel Main

To Supply the North Side of Pittsburgh with Pure Water Main Has Been Connected with Filter Plant

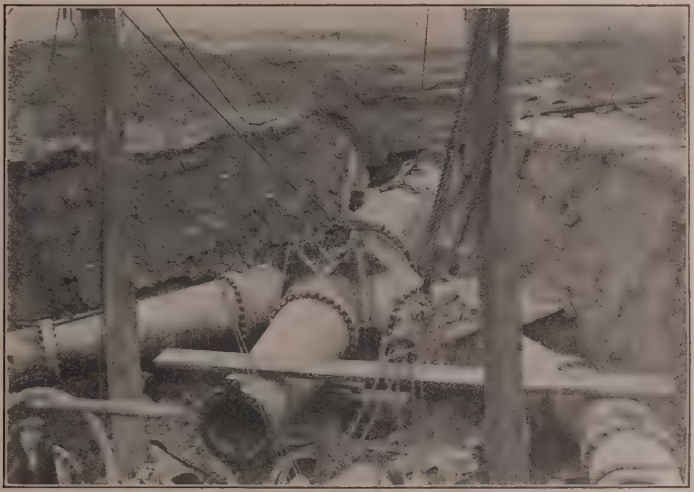
THE North Side, Pittsburgh, is at present supplied with unfiltered water through a 60-in. steel main running up to a pumping station on the Allegheny River. In its course it passes the filtration plant at Aspinwall, but has no connection thereto. In order to supply the North Side with filtered water the 60-in. main has just been tapped at the filter plant.

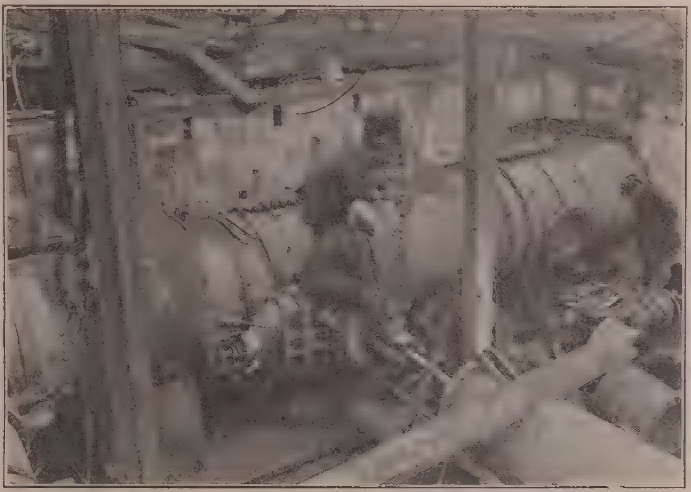
Since the available storage on the North Side was not sufficient to allow the main to be thrown out of service long enough to connect the 60-in. line by a bend to the 60-in. main from the filtration plant, recourse was had to three 36-in. taps on the steel line, all made under pressure by means of tapping machines built by the A. P. Smith Manufacturing Company, of East Orange, N. J. This connection may, if desired, be replaced later by a 60-in. riveted steel bend. This would considerably reduce the friction loss.

In order to make the connection, the 60-in. main was uncovered and a concrete chamber, 16 ft. 6 in. by 37 ft. 6 in. in plan, was built around it. The main was then surrounded with three heavy cast-steel sleeves made in two parts, held in place by heavy bolts. These sleeves were placed 5 ft. 10 in. apart on centers, as shown in the illustrations. Special pains were taken to secure as tight a fit as possible between the sleeves and the pipe, for it was feared that the flexibility of the steel main would allow readily of distortion and thus make tight calking difficult. Even with the precautions taken, however, no less than 2500 lb. of lead were required for each sleeve. The expected distortion, however, did not occur and no difficulty was found in making a tight joint.

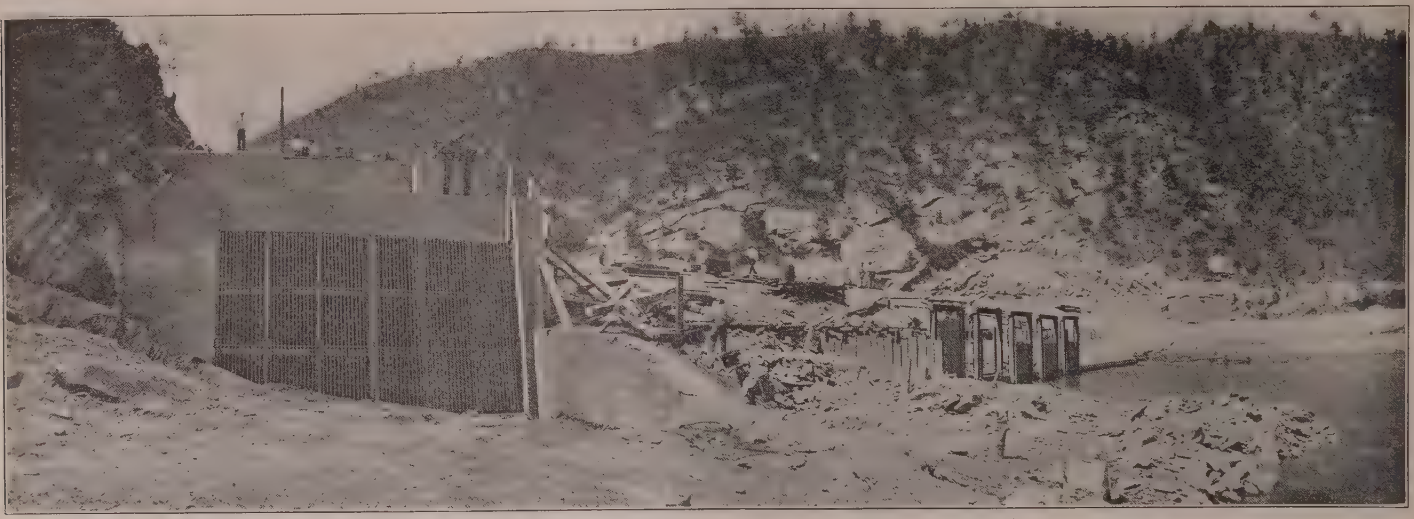
The method of procedure is plainly shown by the photographs. The sleeve, it will be noted, is fitted with a flange, to which is bolted a 36-in. gate valve. The tapping machine, consisting of a tool for cutting a 36-in. disk out of the steel main, and the gearing necessary to operate the cutting tool are then bolted on outside the valve. The valve is opened, the cutting tool entered, the disk cut out and drawn with the tool into the chamber of the tapping machine, and the valve then closed. The tapping machine is removed and the reducer between the valve and the three-way branch inserted. The three-way branch is a steel casting connecting to the 60-in. main from the Aspinwall plant.

The work was done under the direction of Charles A. Finley, superintendent of the Bureau of Water, Pittsburgh.


Three-Way Branch of 60-Inch Main


Three 36-Inch Taps in a 60-Inch Main

PLOWING UP worn brick pavement, instead of employing a large gang with picks and crowbars, was the experiment successfully tried by a contractor in Louisville, Ky., according to the "Brick and Clay Record." The brick in the 1-mile stretch to be repaved was worn too thin to be of use in building construction, and its use as pavement foundation was prohibited. The contractor used a rooter plow, with a heavy point in place of the regular blade, propelled by a 10-ton steam roller—an unusual procedure explained on the ground that the contractor owned no traction engine except the roller, which was idle at the time. This combination tore up half a block of street in from two to three hours, with a force of four or five men.



Upstream Side of Flume Intake and Stone-Filled Crib Dam on Which Wooden Deck Is Being Placed

Developing Electric Power under 250-Foot Head in Tennessee

Second of a Series of Plants on Ocoee River, Involving 4½-Mile Flume and Siphonic Spillways at Forebay

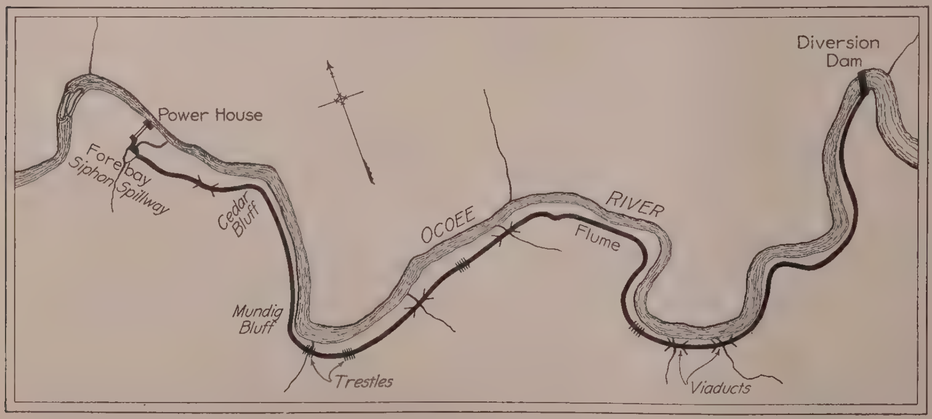
By WILLIAM P. CREAGER
Assistant Hydraulic Engineer, The J. G. White Engineering Corporation

THE Tennessee Power Company owns or controls a number of steam and hydroelectric-power plants in eastern Tennessee, as well as transmission lines for supplying power to various utilities in Chattanooga, Cleveland, Knoxville and Nashville, Tenn., and Rome, Ga. The water-power resources of this company are located principally on the Ocoee River above Parksville, Tenn. The total fall of this river from Ducktown down to Parksville, a distance of 26 miles, is about 710 ft. Additional power sites as well as storage sites are located in the reaches of the river above Ducktown. Of the fall between this town and Parksville 110 ft. have been utilized by Development 1, a 27,000-hp plant at Parksville, which was described in the Engineering Record of June 22, 1912, page 676. The head is developed by the construction of a 110-ft. dam at Parksville. Its backwater extends for about 7½ miles to a point a short distance below the tailrace of Development 2 and forms a body of water known as Lake Ocoee. Development 2, which has been completed recently, utilizes a fall of 272 ft. in about 4½ miles of the river above the pond of Development 1 and provides for an ultimate installation of 30,000 hp. It is the intention of the power company to develop the additional power possibilities above Development 2 as soon as the increasing demand for power is apparent.

HYDROGRAPHIC CONDITIONS

THE Ocoee River rises in the Appalachian Mountains at an elevation of about 3000 ft. above sea level. The upper part of the watershed has an average annual rainfall exceeding that of any other part of the United States with the exception of a small district in Oregon and Washington. The mean annual rainfall on the entire watershed is 55 in. The drainage area tributary to Development 2 is 515 sq. miles. Most of this area is heavily wooded, but a section of about 28 sq. miles near Ducktown, having been denuded of vegetation by fumes from copper-smelting plants in that vicinity, is rapidly eroding. For this reason considerable quantities of silt are being carried by the river during floods.

The U. S. Geological Survey has maintained gaging stations on the Ocoee River at Blue Ridge, Ga., since 1899 and at McCays, Tenn., since 1903. The Tennessee Power Company has also maintained a gaging station at Parksville since 1910. The discharges at these stations, having been corrected for area and physical features of the drainage basin to suit the conditions at Development 2, indicate an average discharge for the years of record of 1310 sec.-ft. and a minimum monthly average of 280 sec.-ft. The maximum flood to be expected has been computed to be about 40,000 sec.-ft., but the development has been designed to pass with safety a discharge considerably in excess of that amount.



Map Indicating General Relation of Diversion Dam, Flume and Power House

Development 2 provides for the utilization of 1200 sec.-ft. This discharge will be available most of the time after contemplated storage reservoirs have been constructed. The total fall in the river between the crest of the diversion dam and normal water surface in the tailrace is 272 ft. At the turbines 250 ft. of head are available for the generation of power.

The diversion dam is located about 12 miles above Parksville. From this dam the water is carried in a flume 4.5 miles along the mountain side overlooking the river to the penstock intake located directly above the power house at the upper end of Lake Ocoee. The present installation consists of two 10,000-brake hp units, but the flume has sufficient capacity to provide for an ultimate installation of 30,000 brake hp. The current is carried directly to the power house of Development 1, where it is cut in to the general transmission system.

DIVERSION DAM AND INTAKE

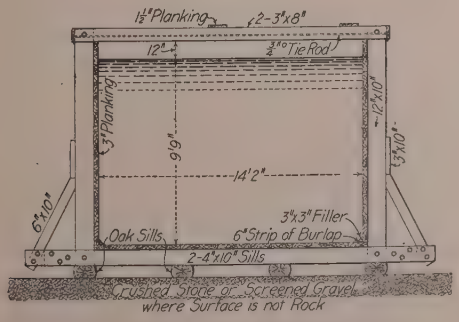
THE diversion dam is a rock-filled crib structure 385 ft. long at its crest. At the maximum section it is 27.5 ft. high and 40 ft. wide at the base, with an apron extending 31 ft. downstream. The dam rests throughout on rock foundation. The crib is composed of 10-in. square sawed pine timbers fastened at intersections with ¾-in. drift bolts. The upstream face is sheeted with a double lap of 3-in. pine planking. The crest is sheeted with a single layer of 6-in. oak planking. The apron is also a rock filled crib similar in detail to the dam. It is 4 ft. high above rock surface. The top covering is composed of 10-in. square sawed timbers laid in contact and drift-bolted to the crib. The north abutment is similar in character to the rest of the dam, but extends 12 ft. above the crest of the spillway. The south abutment is the flume intake. In the dam 1,130,000 ft. b. m. of timber were used.

The flume intake is a reinforced-concrete structure having an entrance waterway 30 ft. wide and 17 ft. deep. This waterway narrows in a distance of about 40 ft. to 14 ft. 2 in. wide by 9 ft. 9 in. deep at the connection to the flume. The intake forms the south abutment to the diversion dam and has its top 12 ft. above the spillway crest. The racks are composed of 4 x ½-in. steel bars spaced 4.4 in. on centers. The discharge through the intake is controlled by a 14 ft. 2 in. wide by 10 ft. 9 in. high steel Tainter gate operated by a hand hoist. For normal water surface in the pond and 1200 sec.-ft. passing into the flume the water surface at the gate is 12 in. below the top of the opening. The water has a velocity of 8.7 ft. per second. For small discharges the gate is nearly closed and the velocity reaches a value as high as 20 ft. per second. This velocity prevails for a distance of about 40 ft. in the flume, then gradually reduces as the velocity head is regained, while the depth increases to that required by the flume to pass the discharge.

Flume

THE flume on the mountain side is a wooden structure designed for a waterway 14 ft. 2 in. wide and 9 ft. 9 in. deep when carrying the maximum required capacity of 1200 sec.-ft. The average slope is 0.64 ft. per thousand, but this slope was increased on the curves and reduced on the tangents. The average coefficient of roughness for use in Kutter's formula was assumed at 0.011. The side boards have a superelevation of 12 in. above water surface in order to avoid overslopping from wind action and to permit the passage of trash. The bents are spaced 4 ft. on centers. At each bent the two 3 x 8-in. beams across the top rest on the side planking, but are bolted to the side posts. Between these two beams, and extending through the upper end of the side posts, is a ¾-in. tie rod with upset screw ends and 4 x 4 x ½-in. plate washers. This rod is designed to take the whole reaction from the top of the side posts. Each side post is composed of 8 x 12-in. long-leaf or 12 x 12-in. short-leaf pine. Two 4 x 10 in. sills support the flooring. The connections between side posts and sills are made by bolting a 10 x 10-in. oak filler block 20 in. long between an extension of the sills and abutting against the bottom of the side posts. In this way the three 1-in. bolts transfer the bottom reaction from the side posts through the filler blocks to the sills, with all compression at the bolts parallel to the grain of the wood. A 6 x 10-in. brace on each side of the flume provides the necessary rigidity. The sides and floor are composed of 3-in. yellow-pine planking dressed four sides and grooved for 1 1/3 x ⅝-in. splines.

Across small ravines the flume is carried on timber trestles. Five of the ravines, however, are too large for this method of support and steel trestles are provided.

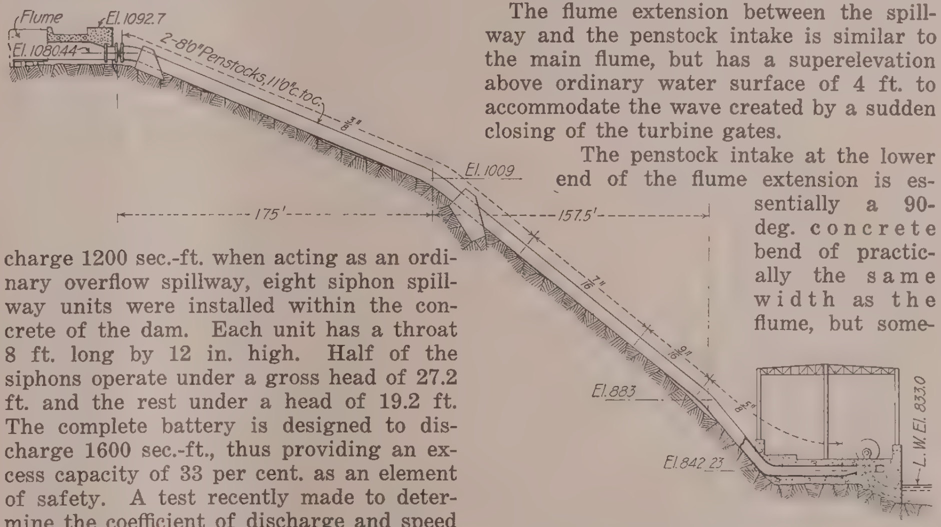


Cross-Section of Wooden Flume

The largest of these trestles has a length of 288 ft. and a maximum height of 150 ft. For a considerable distance at the upper end where the foundation is rock the side boards of the flume have been left low enough to provide for an emergency spillway in case the Tainter gate at the flume intake is allowed to pass more than the desired maximum discharge. For the present turbine installation the spillway has been adjusted to limit the flume discharge to 800 sec.-ft., but when the complete installation is provided the spillway crest will be raised to correspond to a flume discharge of 1200 sec.-ft.

Siphon Spillway

IN order to take care of the surplus discharge, when part or all of the load on the station is suddenly thrown off the turbines, a spillway is provided near the lower end of the flume about 300 ft. above the penstock intake. A concrete spillway dam about 90 ft. long was constructed across a small ravine. As the available length of crest was not sufficient, with reasonable fluctuations in the water surface, to discharge 1200 sec.-ft. when acting as an ordinary overflow spillway, eight siphon spillway units were installed within the concrete of the dam. Each unit has a throat 8 ft. long by 12 in. high. Half of the siphons operate under a gross head of 27.2 ft. and the rest under a head of 19.2 ft. The complete battery is designed to discharge 1600 sec.-ft., thus providing an excess capacity of 33 per cent. as an element of safety. A test recently made to determine the coefficient of discharge and speed of priming of these siphons resulted in their fully meeting the expectations of the designers.

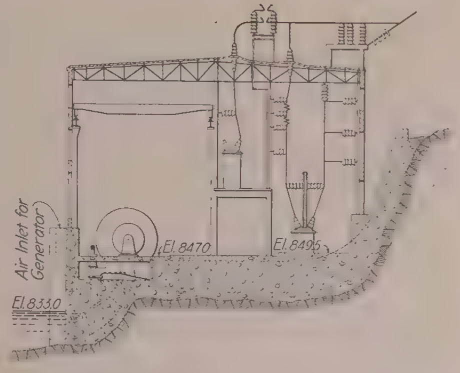


Arrangement of Penstock and Power House

The spillway dam rests mostly on a foundation composed of earth containing large boulders, but good rock foundations were encountered at both ends. On account of the boulders it was impossible to drive sheet piling, and watertightness was secured by excavating a trench under the upstream side of the base 6 ft. wide and about 15 ft. deep, and refilling it with concrete. The portion of the dam resting on earth foundation was considerably widened in order to avoid unequal settlement with resultant tendency to tip. This portion of the base is reinforced with 1¼-in. square bars spaced 12 in. on centers extending well into the concrete of the cutoff trench. In order to make these bars effective a slab 8 ft. thick constructed above the foundations contains no building joints. At its maximum section the dam rests on earth foundation and is 40 ft. wide at the base and about 38 ft. high.

In order to prevent scouring of the bed of the ravine the discharge from the siphons is carried about 300 ft. through a 6 x 8 ft. covered wooden waste flume to a point where the rock outcrop extends completely across the ravine.

A trench, about 10 ft. deep and extending upstream from the dam, intercepts sand and gravel carried along the bottom of the flume. This trench is cleaned through a 48-in. sluiceway through the dam. The flume discharges directly into the forebay formed by the dam. The current from the flume is directed toward the sand sluice by means of a submerged wooden partition supported at intervals by rock-filled cribs. A timber boom across the forebay deflects floating material to a 4-ft. trash chute through the dam controlled by stoplogs.

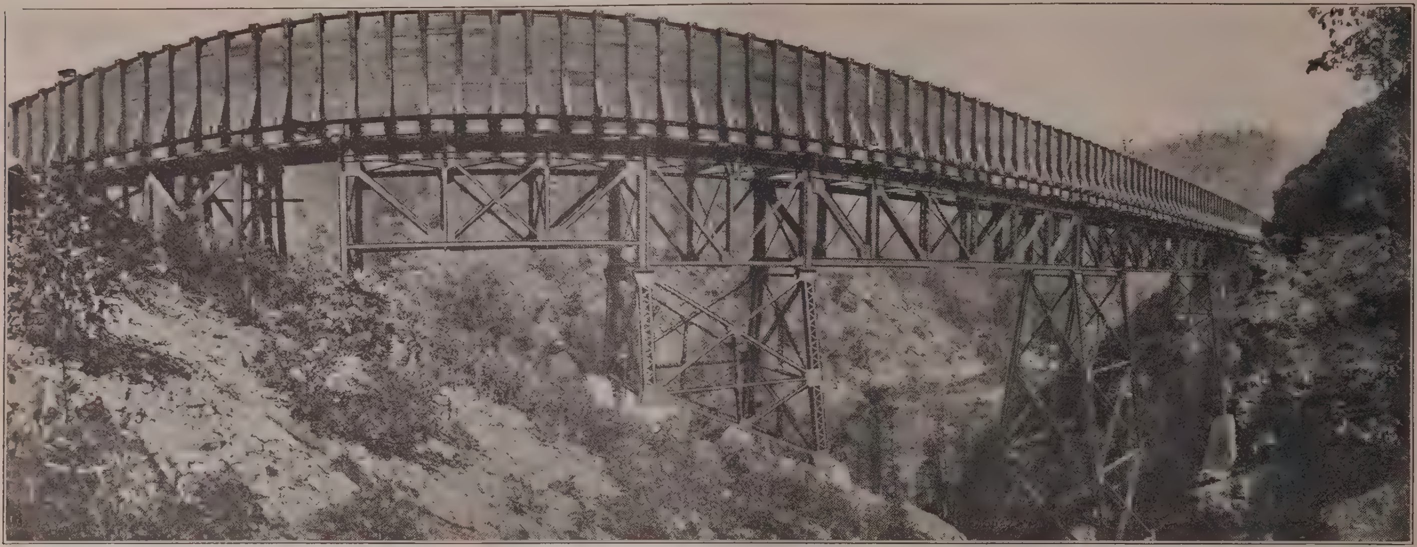


Sectional Elevation of Power House

The flume extension between the spillway and the penstock intake is similar to the main flume, but has a superelevation above ordinary water surface of 4 ft. to accommodate the wave created by a sudden closing of the turbine gates.

The penstock intake at the lower end of the flume extension is essentially a 90-deg. concrete bend of practically the same width as the flume, but somewhat deeper. It is about 40 ft. long on the center line. On account of the sharpness of the bend, with liability of water piling up on one side, the intake is divided into three compartments by two 11-in. reinforced-concrete walls and entirely covered with a reinforced-concrete slab. In this way the water for the three penstocks is equally divided before passing into the intake.

Two penstocks have been provided for the present installation. Provision has been made for the erection of one additional penstock when the future turbine is installed. The pipes are 470 ft. long and 8 ft. in diameter between the intake and the power house, but are reduced to 5 ft. 5½ in. where they flange to the volute casings of the turbines. The thickness varies from ⅜ to ⅝ in. They are designed to withstand the maximum static head plus 25 per cent with a factor of safety of 4. Immediately below the intake in each penstock is installed an 8-ft. butterfly valve. The valves are both motor and hand-operated. The motors are controlled from a switchboard in the power house and are capable of completely closing the gates in 20 seconds. These valves are located just above a 23-deg. bend in the penstocks. A wedge-shaped section formed by the bend is provided with flanged joints and can be easily



Steel Towers and Trusses Carrying 4½-Mile Flume of the Ocoee River Development 2 Across Declivity

removed should it be necessary to detach the butterfly valves. The air inlets below the valves are 18 in. in diameter. The penstocks are secured in position by two large concrete anchorages, one just below the butterfly valves and the other midway down the hill.

INSTALLATION

EACH of the two main generating units consists of a 9375-kva, 6600-volt, three-phase alternator, direct-connected to a 10,000-hp horizontal turbine running at 360 r. p. m. A 120-kw, direct-current exciter is connected to each generator shaft. A 50-kw motor-generator set is provided to supply the direct current for the crane. Two 9375-kva oil-insulated, water-cooled transformers have been provided for the present installation. These transformers raise the potential from 6600 volts to 66,000 or 120,000 volts. They are at present operating at 66,000 volts. The complete electrical installation was furnished by the General Electric Company.

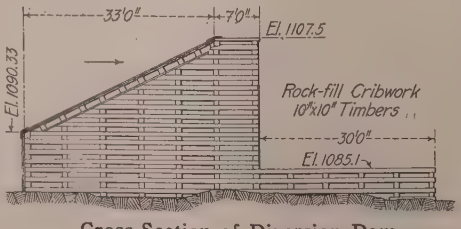
The turbines were furnished by the I. P. Morris Company. Each turbine was designed to develop 10,000 hp at 360 r. p. m. under 250-ft. effective head. They are guaranteed to deliver 10,000 hp with 84 per cent efficiency, 9100 hp with 87 per cent, 7500 hp with 84 per cent, and 5000 hp with 75 per cent efficiency.

They are of an inward-flow, double-discharge reaction type, commonly known as high-head Francis. Each turbine has a horizontal forged-steel shaft, one double-discharge bronze runner with finished vane surfaces, a single cast-iron volute casing and two plate-steel draft tubes. The penstock is attached to the volute casing below the floor. The water passes around the casing and, immediately upon entering the runner, is divided into two parts, each leading to a separate draft tube. The gate rigging is located entirely outside the casing and is connected to the cast-steel guide vanes by forged shaft extensions of the guide vane castings, which pass through bronze-bushed stuffing boxes in the casing. The runner and shaft are carried by two self-aligning, babbitted, ring-oiling, generator-type bearings. The unit, being of the double-discharge type, is hydraulically balanced. However, lignum-vitæ thrust bearings, supported inside the draft elbows, are provided to take care of any unbalanced effect which might occur and to provide definite horizontal alignment of the runner.

Each turbine is provided with a friction brake designed to bring the rotor from full speed to dead stop in five minutes when the turbine gates are closed and the unit is running at normal speed. The brake acts upon a cast-iron pulley keyed to the outboard end of the turbine shaft. The brake band is cast steel equipped with hard-maple friction blocks. Each turbine is controlled by an I. P. Morris oil-pressure governor. The speed of the unit can also be controlled by a hand-operating mechanism used in conjunction with the oil pressure from the governor accumulator tanks. For starting the unit a hand oil pump has been provided to supply the necessary pressure for opening the turbine gates by the hand control. A special speed-limiting device is located on the turbine shaft coupling. This device consists of a pair of flyballs adjusted so that when the speed rises beyond a critical point a spring is tripped, thus operating a valve admitting oil pressure to the pilot valve, resulting in the closing of the turbine gates. A "Y" branch has been connected to the bottom of each volute casing below the floor leading to a 24-in. relief valve which can be adjusted to operate automatically when the pressure within the penstock rises 25 per cent above normal.

The tailrace is very simple in character, consisting of a channel about 80 ft. wide and 60 ft. long extending from the face of the power house at the draft tubes to the main channel of the river. Low water is 10 ft. deep at the power house and 5 ft. at the river channel.

All of the engineering and construction work in connection with the development was done by The J. G. White Engineering Corporation, of New York. The preliminary engineering work required some time, and the actual construction work was started in May, 1912. The work proceeded with rapidity, and in December, 1913, nineteen months from the date of starting, the development was completed and power generated.



Cross-Section of Diversion Dam

Railway Tunnel under the Tyne

A PROJECT for driving an electric tube railway under the River Tyne in England between North Shields and South Shields is contemplated, according to the London "Times." The districts to be served directly have a population of 160,000, but there is, within a radius of 10 miles, a population exceeding 1,000,000. The Tyne at this point is about 1200 ft. wide, and the general levels of the ground at each side are considerably above the river level, so that the approaches to the ferries are excessively steep.

The railway will be ¾ mile long, of standard gage, single track and nearly straight. At North Shields the platforms will be 110 ft. below surface, and electric elevators in 32-ft. iron-lined shafts will be provided. The platforms at South Shields will be about 25 ft. below the street level, and will be reached by stairs and inclines. At each station the platforms will be placed on both sides of the trains, so that passengers will alight on the opposite side of the train to that on which they enter. A single train working backwards and forwards will maintain a six-minute service.

The platforms at North Shields station will be in a tunnel of 23 ft. internal diameter, and constructed in the clay strata by means of a shield. The tube under the river will have an internal diameter of 11¼ ft. The strata here consist chiefly of sandstone, clay, gravel and gray shale, and the work will be carried out under compressed air. The remainder of the work will be carried through brown stony clay. The tunnel for a length of 600 ft. under the river will be laid at a maximum depth of 68 ft. below high water, and will rise toward North Shields on a grade of 3.62 per cent.

NITROGEN OVER 1 sq. mile of the surface of the earth is estimated to be sufficient to supply the whole world with fertilizer for fifty years at the present rate of consumption. To produce nitrate fertilizer from the air to an amount equal to that now used by the civilized world would require an investment of $860,000,000.

Proposed System to Supply Water to Richmond, Cal.

Plan to Use Water from Sacramento River, Convey It in a 36-Inch Pipe Conduit 45 Miles Long and Filter It En Route

RESULTS of twelve months' investigation of the best method of developing a suitable water supply for the municipality of Richmond, Cal., have just been published in a 211-page report which goes exhaustively into the various elements entering the problem. Three available sources are mentioned, namely: Wells within the district, surface runoff within and adjacent to the district and filtered water from the San Joaquin or Sacramento River. The last of the three was recommended by the engineers and has been adopted by the board of water commissioners of the municipality, who have decided to call for an election to authorize a $2,500,000 bond issue.

In the district to be supplied the present water consumption is about 71 gal. per capita per day for domestic use and 123 gal. per day for manufacturing purposes—a total of 194 gal. per day. It is estimated that this abnormal industrial consumption will increase slowly for eight years and then decrease until 1955 when the consumption is estimated at 150 gal. per capita per day.

The water of the Sacramento River was found to have less sewage pollution, and to be clearer and softer than that from the San Joaquin River, and a point near Toland's landing on the former stream was selected for the intake because of low chlorine content of the water, solid foundations for structures, and a permanent channel course. The quality of the water was reported perfectly satisfactory when properly filtered.

GENERAL FEATURES

THE general features of the proposed works comprise a pumping plant at Toland's landing and a 45-mile conduit line to Richmond, where two reservoirs with a total capacity of 92,000,000 gal. would be so located as to supply the city through a gravity distribution system. At Martinez, about 15 miles from Richmond, there would be a booster pumping plant, duplicating the installation at the intake, and also gravity-type mechanical filters, and a storage reservoir.

The pumping plants are to consist of horizontal, centrifugal, motor-driven pumps with an ultimate capacity of 22,500,000 gal. per day. Emergency gasoline engines are specified. Detailed analyses lead to the conclusion that, considering first cost as well as pumping cost, a 35-in. pipe would be the most economical diameter. It is therefore planned to use a 36-in. wood or riveted-steel pipe. This will cross the Sacramento and San Joaquin Rivers to the south side of the latter, and follow along the south shore to the filters and storage reservoir near Martinez, continuing thence to Richmond either along the south shore or inland through a 3-mile tunnel. The tunnel route would be shorter and would involve less pumping, but would eliminate possible water consumers along the shore line. The conduit is to have a capacity of 15,000,000 gal. per day, while the filters at Martinez would have an ultimate capacity of 18,000,000 gal.

If the conduit from Toland's landing is built as planned, it is estimated that during the life of the project water can be sold at an average cost of 15 or 16 cents per 1000 gal., although during the time that the demand was far below the capacity, the cost might be as high as 20 or 25 cents. These costs were estimated for a forty-year bond issue of $2,500,000 at 5-per cent interest, one-fortieth retired each year, and include all fixed charges, interest and depreciation, as well as operating and distributing expenses. The estimated cost of the conduit and distributing system for a supply of 12,000,000 gal. per day and including right-of-way if the shore route were used, was $3,803,862 if wood-stave pipe were used for all pipes more than 18 in. in diameter, $4,246,198 if riveted-steel pipe were used and $5,799,217 if cast-iron was selected.

The report, and the detailed designs for all parts of the project, were prepared by Haviland, Dozier & Tibbetts, civil engineers, of San Francisco.

Book Reviews

The "Proceedings" of the thirty-third annual convention of the American Waterworks Association, held at Minneapolis, June 23 to 27, 1913, is a book of 750 pages filled with a great variety of data valuable to the waterworks engineer and superintendent. Year by year the character of the proceedings has attained a higher plane, so that the record of the meetings is now invaluable to any complete engineering library. The discussions in this volume are especially numerous. (Troy, N. Y., American Waterworks Association, J. M. Diven, Secretary.)

The "Proceedings" of the twenty-third annual convention of the American Railway Bridge and Building Association, held at Montreal in October, 1913, are published in a 375-page paper-bound volume. Reports on water supply, track scales, sewers and drains, motor cars for bridge gangs, structures for supporting temporary tracks, snow fences, preservation of timber, cattle guards, fire-resisting coatings for timber and protection of concrete in sea water occupy most of the book. More than 150 pages are devoted to the report of the committee on track scales, which appears to be the most comprehensive and valuable discussion of the subject yet published. Of special value also are the data on water supply and on preservation of timber, the latter dealing in part with phases not ordinarily treated. (Chicago, American Railway Bridge and Building Association, C. A. Lichty, Secretary. $1.)

"Report on the Water Powers of Tennessee" is the title of Bulletin 17 of the State Geological Survey, which contains 136 pages and twelve plates, prepared by Prof. J. A. Switzer of the University of Tennessee, hydraulic engineer of the Geological Survey. The report takes up the economic aspects of Tennessee water power, the relation of the State to its water-power resources, arguments for water-power legislation, recommendations, available water power, present development, projected development and the report of the Doe River survey. Appendix A is a digest of the laws of other States with reference to water power; Appendix B, a digest of those of other countries. (Nashville, Tenn., State Geological Survey. 8 cents for postage required.)

Industrial education is to-day occupying in the minds of executives a place only second to the legislative measures designed to regulate business. The awakened interest is evidenced by the activities of men and organizations fathering continuation schools and promoting vocational education. One of the most valuable pieces of literature so far offered is a 440-page volume containing the proceedings of the organization meeting and of the first annual convention of the National Association of Corporation Schools. Though but little more than one year old, the association has taken a place in the educational movement which augurs a large influence in the near future. The character of the proceedings of the Dayton convention is such that no industrial manager can afford to be ignorant of them. In addition to reports of the committees on selling, apprenticeship, engineering and office-work schools, there are papers on the various phases of the educational work being carried on by such organizations as the National Cash Register Company, the Yale & Towne Manufacturing Company, the General Electric Company, the Pennsylvania Railroad, the Burroughs Adding Machine Company, and the New York Edison Company. (New York, National Association of Corporation Schools, 124 West 42d St. $2.50; postage extra.)

THE PANAMA CANAL. By Frederick J. Haskin, Author of "The American Government," etc. Cloth, 5½ x 8½ in., 386 pages, 54 illustrations. Garden City, New York, Doubleday, Page & Company. $1.25 net.

A general description of the canal itself, of the method of organizing the working forces, of the types of machinery used, the records made, the mode of government, the sanitary measures, the antecedent political history, the fixing of the tolls and of the effect of the canal on the world's commerce is given in this volume. It is designed primarily for the information of the layman and is, therefore, divested of technicalities. The book was read and corrected under Colonel Goethals' direction, and within its limits can be taken as reliable.

INDUSTRIAL EDUCATION; ITS PROBLEMS, METHODS AND DANGERS. By Albert H. Leake, inspector of technical education, Ontario, Canada. Cloth, 5½ x 8 in.; 200 pages. Boston, Houghton Mifflin Company. $1.25 net.

This volume was awarded the Hart, Schaffner & Marx prize for economic essays for 1912. It is a very comprehensive review of the whole problem, based not on speculation but on careful study of the present educational system in the United States and Canada, of various types of schools proposed as remedies or established both in North America and in Europe. The author believes that a reform of our elementary system of education is necessary. This is not a new idea, but much data are brought to bear on the methods proposed as a substitute. A chapter on "The Problem in its Various Aspects" gives an excellent view of the conditions crying for remedy. Then in turn are taken up the revitalization of the elementary school; the present place and future of manual training; factory and special industrial schools; continuation schools; apprenticeship; vocational guidance, and, finally, dangers arising from misinterpretation of foreign industrial educational schemes. The work is of much value for reference and as a summary to those who appreciate and are following this very important problem.

The Truth About the Railroads. By Howard Elliott, chief executive officer of the New York, New Haven & Hartford Railroad and of the New England Transportation Lines. Cloth, 5 x 8 in., 260 pages. Boston, Houghton Mifflin Company. $1.25 net.

The standing of the author of this work as an able railroad executive, thoroughly familiar with all phases of the great transportation problems is guarantee that the book is worth the careful consideration of those who wish to know the railroad's side in the many controversies constantly arising with reference to our carriers. The several chapters were first written as addresses to be delivered on various occasions and in various places. The chapters deal with co-operation between the railway owner, the railway employee, and the railway user; the individual, the corporation and the Government; the conservation of railway service; rate-making and the Government; the relation between the farmer and the railroad; agriculture, banking and the carrier; transportation in New England; public opinion—its effect on business. An introductory chapter puts admirably the harassing conditions to which the railroads are subjected by the unscrupulous section of the press, and the thousands of laws of cities, States and the nation. Where railroad officers have been at fault in not cultivating properly the public the fault is fully admitted, and suggestions made for improving the relations between the railroads on the one hand and the press and public on the other. The various subjects are treated with sound common sense and with a clarity and grasp that carry conviction. The chapter on rate-making and the Government, being a statement before a Congressional committee in 1905, is of especial interest. It discusses in detail the factors which affect rate-making.

The Future of the Working Classes. By Roger W. Babson. Boards, 5 x 8 in.; 76 pages. Boston, Babson Statistical Organization, Inc. $0.50.

Arguing from an economic basis Mr. Babson concludes that the improvement of the working classes will come only through increasing their efficiency, or, in his way of putting it, enlarging their capacity for more responsible work. While in his conclusion most people will agree it is doubtful whether they will be able to see that conclusions follow logically from the premises he lays down. While in Mr. Babson's own mind the steps may be clear, it will require more detailed discussion than is given in this little book to show others the connection. The method he proposes for increasing the capacity of the workers is economic education, in which work of shop grade, and for which the child, or his parent, will be paid, starts at the age of nine. But two hours are given to work at the start, gradually increasing to a full eight hours at the age of 22. This conception, while more elaborate, is fundamentally that of the continuation school. Mr. Babson would start actual work earlier and continue it later. With the great interest now shown in continuation, vocational and corporation schools it will be surprising if expansion along the lines of better economic education does not come rapidly. That it will go to the length of Mr. Babson's plan, however, is doubtful. His belief that the financial powers are not in favor of better education of the workers is open to much discussion. If it is taken sweepingly it is untrue; if applied to certain industries, the charge, unfortunately, must lie, but even then condemnation—and it is deserved—should be accompanied by a full understanding of the special conditions of the industry.

Elementary Graphic Statics. By John T. Wight, Lecturer in Machine Design and Prime Movers, Heriot-Watt College, Edinburgh. Cloth, 5 x 7 in., 227 pages, 135 illustrations. London and New York, Whittaker & Company. $1.25 net.
(Reviewed by Frank H. Constant, Professor of Structural Engineering, University Minnesota.)

This book was written to fill the author's needs while lecturing on the subject of graphic statics to first-year students in the Heriot-Watt College, Edinburgh. It is, accordingly, confined to the simpler problems met with in engineering construction, and treats only of graphical processes. It is a small volume of 227 pages, with 135 illustrations. The chapter headings indicate the scope of the work. These are the introduction (with definitions of forces, etc.), composition and resolution of forces, simple practical problems (of the crane type), composition of non-concurrent forces, bending moment and shearing force diagrams, beams with rolling loads (confined to one or two concentrations and uniform moving loads), symmetrical and unsymmetrical roofs for various kinds of loading, braced beams and girders, centers of gravity, neutral axis, resistance figures, moments of inertia and retaining walls.

In the last chapter only trapezoidal masonry walls are considered. The earth pressure is obtained by graphical constructions, using Rankine's, Coulomb's and Rebhann's theories. The treatment is elementary and brief.

The book is clearly written. It is intended to meet the needs of those who have had no preliminary knowledge of mechanics or advanced mathematics. It might prove quite serviceable in extension or correspondence work in elementary structural courses. A very valuable feature is the list of practical problems at the close of each chapter. These problems illustrate the principles of the text and show their application to actual every-day construction. The typographical work in text and cuts is clear and inviting.

Railway Misrule. By Edward Dudley Kenna. Cloth, 5 x 7¾ in.; 163 pages. New York, Duffield & Company. $1.25 net.
(Reviewed by W. J. Cunningham, Assistant Professor of Transportation, Harvard University, Cambridge, Mass.)

This interesting and well-written little book contains a review of the railroad situation from the days of the Granger movement until the present time, and offers a constructive program for the elimination of railroad difficulties.

As Mr. Kenna sees the problem it is essentially one of finance. The transportation machine is deficient because it has not kept pace with the ever-increasing traffic. Just now there is a stoppage in this growth, but with the resumption of normal conditions railroad facilities will be overtaxed. To provide the needed facilities will call for the enormous sum of $30,000,000,000 within the next twenty years. And just now, with confidence shaken in the security of railroad stocks and bonds, and with operating expenses increasing because of higher wages, higher cost of materials and legislative requirements with respect to service, equipment and safety, the constantly narrowing net return makes it practically impossible to attract capital in large amounts. Advances in rates will not afford a permanent solution because of the inherent differences between the strong and the weak lines. The commissions must choose between fattening the already prosperous roads by granting higher freight rates, or starving their weak competitors by refusing to sanction the advances needed by the weak lines to keep them from bankruptcy.

The author's solution is a repeal of the Sherman law in so far as it has been made to apply to railroads. Allow the strong roads to absorb their weak competitors. Make a compact system with a monopoly for each district, and cut out the large wastes caused by competition. Route the through traffic over the line best equipped to handle it expeditiously and economically and allow the poorly located and poorly built lines to handle local business only. The losses on the latter would be offset by the greater net revenue from the well equipped lines.

The idea is not altogether original. It has been suggested in connection with recent wage arbitration, when the weak roads contended that the application of the standard rates of the prosperous roads would bankrupt the poor lines. The tendency of regulation at this time is squarely against the suggestion, as is evidenced by the recent dissolution of the Harriman lines and the more recent case of the New Haven road. A practical difficulty not referred to is the inevitable and forceful protest of the cities, towns and villages located on the lines now having through service which would be relegated to the "local" class between competing centers.

The final chapter on Government ownership appears to have been added as an afterthought. The reader is not prepared for the author's endorsement of that policy, as he has been led to believe from the introduction and from the succeeding chapter that the author's one solution was his plan of elimination of competition and a monopoly by one system in each geographical section, with strengthened control by the commissions over rates, service, wages and other operating features. To have Government ownership cited as another solution is illogical and unconvincing, when considered with the context.

All of Mr. Kenna's suggestions, however, are interesting and deserve careful consideration by students of transportation problems. The book is written in a clear style and the author has a good grasp of his subject.

WATER CONSUMPTION in Chicago was 203 gal. per capita in 1913, a decrease of 15 gal. over 1912. The heads pumped against were 115.84 ft. and 120.24 ft. respectively in 1912 and 1913 according to the annual report of the Board of Public Works. Meters have been placed on the discharge from all stations and the average slip determined to be 9 per cent. The maximum daily consumption occurred on June 3, when 256 gal. per inhabitant were pumped, a quantity 20 per cent in excess of the average. The total cost of pumping per 1,000,000 gal. 1 ft. high was 3.42 cents in 1913 and 3.76 cents in 1912. Of this cost the fuel was 1.66 and 1.87 cents respectively for the two years, the average price paid for coal for the major stations being $2.19 in 1913 and $2.36 in 1912. The efficiency of operation increased slightly from 15.1 lb. of coal per million gallon-feet in 1912 to 14.3 lb. in 1913. The total cost of pumping by steam and by electricity at the Twenty-second Street station for a similar unit was 4.04 cents.

Letters to the Editor

The War Against the Waves

Sir: Dr. Corthell's article on "The War Against the Waves" in your issue of April 11, page 408, is most interesting and instructive. It is especially valuable because of the assurance it gives to engineers that the problem of shore protection has been solved by using the mighty forces of the water to shape and form the protective barrier by persuasion, as it were, rather than by trying to back the immovable body against the irresistible force. His reference to the conservation of the shores through Federal agencies and his mention of the Coast and Geodetic Survey as well fitted to gather the data are doubtless based on his knowledge of the work done in the Survey by Mitchell at Sandy Hook and Martha's Vineyard and by Marindin at Martha's Vineyard, Nantucket and Cape Cod, and also that by Harris on tides and tidal currents.

To carry out his suggestion would require an extension of the activities of the Survey along lines already followed in the past and indicated in the law as one of its functions. The work, however, would have to be pursued in a much more comprehensive and continuous manner than has hitherto been possible with the limited force and means at the command of the Survey. In all probability a co-operative scheme between State and Federal Governments could be evolved which would make a fair division between matters of local and National import in the same manner in which the Newlands-Broussard bill (Senate bill 2739) suggests co-operation between State and National agencies.

O. H. Tittmann,
Superintendent, U. S. Coast and Geodetic Survey.
Washington.

The Aneroid for Railway Reconnoissance

Sir: In an article in your issue of March 14 last describing features of the location of the Alberta Central Railway, there is a description of methods of reconnoissance by aneroid barometer which concludes with the statement "that the result [of the surveys] is sufficient to show that accurate work can be done by the method explained." It is further stated, however, that the final error in leveling was only 20 ft., although intermediate errors of more than that (and in the estimation of the present writer considerably more) are admitted, and in view of this it might have been more nearly correct to say that the aneroid will give certain approximate information which is of value under certain circumstances.

It is apparently intended to show that the aneroid is of value, by stating that the route selected after a preliminary survey did not vary from that selected by the engineer who made the reconnoissance, though the present writer fails to see the connection in so far as this sheds any light on the present value of the aneroid as an instrument for railway reconnoissance.

The first part of the route is apparently over more or less rolling prairie country, which, by the bye, often offers some of the most difficult problems in location, but where nothing short of fairly accurate levels are of value, where anything approaching the maximum of such a light rate of gradient as 0.4 per cent. is required. The general route through such a country as this is fixed within certain fairly wide limits by the direction, and the determination of details cannot be made with anything less accurate than a transit or level, or both. Any good railway engineer of sufficient experience could select a general route through such country either with or without the aneroid. The profile given is, of course, of too small a scale to be of any value for definite location, although it is of interest as showing in a general way what the country looks like.

On the second section, where there seem to be long stretches of supported line at the maximum grade, levels, even though they be accurate, are of little value without some fairly definite information as to distance and direction. This latter apparently was obtainable with sufficient accuracy from the township and section lines, but the present writer feels that this is altogether too flimsy a structure on which to base an opinion of the feasibility, or otherwise, of a long railway line to be built with low rates of ruling gradient.

The article seems to show that the more detailed surveys proved that the reconnoissance was well made, and the writer does not wish for a moment to allow the inference that it was not. He does wish to point out, however, the fact that the aneroid, no matter how carefully used, is unreliable, as indeed is stated in the article where the author refers to "the constant vigilance necessary to avoid errors when using the aneroid, due to its contingent variations and vagaries," and the question, therefore, naturally arises if it is worth while to use such an instrument when a quick stadia reconnoissance can be made at practically the cost of transporting five men (three engineers, a cook and a teamster) across the country. The writer believes from considerable experience that a rough stadia reconnoissance can be made as cheaply as can a reconnoissance with an aneroid. In any event, supposing it costs 50 per cent more, which is an outside figure, the resultant gain is well worth it, as the stadia not only gives reasonably correct elevations (with no intermediate vagaries), but also distance and direction, both of which are really essential, especially on long supported lines.

F. Lavis,
New York. Consulting Engineer.

[Copy of Mr. Lavis' letter was submitted to J. Grant MacGregor, chief engineer of the Alberta Central Railway and author of the article to which Mr. Lavis refers. Mr. MacGregor's reply follows.—Editor.]

Sir: The criticism by Mr. Lavis of the use of the aneroid for railway reconnoissance as described calls for further elucidation of facts by way of comparison. It should be born in mind that the object the writer had in view in the article on the Alberta Central Railway was to point out that of two methods of using the aneroid the one adopted appeared to give the more satisfactory results.

To my mind this is all that can be claimed for the statements made, as they did not pretend to compare the accuracy of the work done by the aneroid, under the most favorable circumstances, with that of other instruments of precision. The fact should not be overlooked, however, that for quick work the aneroid is a very important and useful instrument if properly handled, particularly where fairly good maps of the country are obtainable. Under such conditions it is not impossible to map out a route for such a low grade as 0.4 per cent. It is hardly reasonable to suppose that "any good railway engineer could select a general route through such country" (rolling prairie) "either with or without the aneroid" and be at all certain of the profile to be developed.

It is to be regretted that in publication much of the information shown on the original profile had to be curtailed, and the scale very much reduced. A few obscurities such as the foregoing, and the absence of detail, may have caused Mr. Lavis to lose faith in the results claimed by me. I agree with him that a rough stadia reconnoissance would in the end be more satisfactory where a low ruling grade is sought. The extra time and cost would be more than compensated by the more reliable work outlined for the location party.

J. Grant MacGregor.
Red Deer, Alta.

[It is quite true, as Mr. MacGregor states, that the original reconnoissance profile contained much information that could not practically be shown on the reduced scale necessary to fit the page of the Engineering Record.—Editor.]

Conditions of Railway Employment

Sir: You ask editorially in your issue of April 4 what is wrong with the railway-engineering departments, and you suggest as the answer low pay and slow advancement. These are not the only causes for the reluctant abandoning of railroading by so many promising men after one year or five. In the matter of pay the railroads can hardly be blamed if in their desperate efforts to make both ends meet they hire their engineering talent as cheaply as possible. Personally I believe that by hiring 20 per cent better men and paying them 40 per cent more, and reducing their number to keep the payroll the same, more and better work would be done. But there are other irksome features that could be eliminated with no expense or risk whatsoever.

One of these is the sense of proprietorship which the company holds. Any fair-minded employee will realize that there are emergencies to meet which demand that all abandon their private interests and pitch in. But that is no reason why if it is intended to send a party out on the road for a month the members are not notified until the evening they are to start, nor does it justify selecting one man of several at quitting time to stay and work that night, without evincing any interest in the relative convenience to him or to some of his associates. The fact is that the railroad considers that it owns the man, and if he entertains any outside interests—social, business, educational or religious—he must expect them to be ignored until he has reached a place of considerable authority.

Another characteristic of the service is the lack of faith which the company has in the ability and judgment of the individual. Granted that the cub fresh from college often needs repeated snubbing, a year or two will season him thoroughly if he is of the right stuff. It may reasonably be years before he is given an executive position, but meanwhile he is assigned individual studies of ever increasing importance. Probably he is not clothed with sufficient authority to go to other offices for pertinent data. Like enough his own superior withholds facts that have an important bearing on the matter. Using all the information available, however, he studies his problem a month or two and makes a careful report. Perhaps his superior rejects parts of it at a glance, without looking into their merits. Perhaps it is received and passed to the man above without comment, and its author never hears from it again, even though it is ultimately adopted. The fallacious idea prevails in the railroad office that the chief, dealing with fifty problems daily, can solve each one in a moment without investigating the facts better than the obscure individual who has concentrated on the one problem—and the latter seldom gets a chance to argue his case.

Even the officers are not immune from this meddling. The designing engineer revises the draftsman's work, the chief engineer rejects some more of it and the general manager finds it so bad that he sends it back. If the organization is large and the matter has been pigeon-holed in several offices no effort is made to return it whence it came, and ultimately it gets into the hands of another draftsman less competent than the first, who will certainly make a mess of it unless he is given time to start anew.

One other influence which, though indirect, is ever active and probably has much to do with driving self-respecting men from railroading, is the chief clerk. To quote from a recent address of Major Charles Hine, "the most pernicious practice on railways is in permitting the chief clerk to sign the name of the boss." As Major Hine says, "we would not construct a mechanical plant with pressure from a 3-in. main going through a 1¼-in. connection to reach a 2-in. lateral," yet that is the prevailing railroad custom. In the name of the chief engineer the chief clerk gives orders to men drawing double his pay, and their chief clerks in turn tell better men lower down what to do. Innocent of technical knowledge and aloof from the real workings of the railroad machine, the chief clerk's hobby is making promises for others to fulfil.

His fine Italian hand is also always found on the payroll. He is a great economist, and he frequently saves his company from bankruptcy by reducing salaries, so that the good men leave. Promises of merited promotion are subject to his blue pencil. To him the men are all mere names, and time and again honest efforts of the engineer in charge to differentiate between the real talent, the ordinary men and the dead wood, and to rectify inequalities on his corps, are thwarted by this small man in a large chair.

New York City. C. L. POWELL.

Fire Risk of Small Frame Railroad Buildings

SIR: It seems to be universal practice with railroads to erect minor cheap, inflammable buildings around car shops and motive-power buildings. Oil houses, storage buildings and rest rooms are often old box cars. The practice should be discontinued, as it not only detracts from the appearance of the plant, but increases very materially the fire risk. If the necessity exists for buildings outside of the main structures, the type chosen should be a fireproof building with steel frame, concrete floors, corrugated iron roof and sides, or a concrete block building, with cement floor, concrete slab roof of light pitch, covered with standard built-up felt, pitch and slag, concrete-block partitions and doors covered with iron.

Requirements for storage, etc., at these plants should, if possible, be anticipated. If careful consideration were given to this when the larger buildings were constructed, these minor structures could be built at the same time or done away with entirely by reserving space in the larger buildings for the uses to which the smaller buildings are devoted.

Storing of lumber in large or small quantities adjacent to buildings should be discontinued, even though such storage may be temporary. A particularly dangerous fire hazard came to the writer's attention not long ago. An oil house and two oil tanks above the ground were completely isolated from each other and from all other buildings. It was quite possible, however, for the fire to communicate itself from one to the other and then to the buildings because the tanks were surrounded by empty oil barrels alongside of a number of box cars. The latter were always standing at this point, and were close to a small storehouse which was connected to the oil house by a platform.

Railroad terminals along the waterfront are usually dangerous fire risks when the floors are a few feet above ground which is inundated at high tides. Frequently holes are cut in the floors and covered by trap doors, which are lifted at intervals in order to dispose of papers and other refuse. Danger here enters because these are not always carried away by the tide. Open spaces are also left along the passageways, and into these refuse is thrown, where it collects until an extreme high tide may carry it away. As many people pass along these thoroughfares it is quite probable that a lighted match may be thrown into this rubbish at any time, starting a fire at a point where it would be very hard to combat and from which it could extend to all parts of the terminal.

T. W. REYNOLDS.

Mount Vernon, N. Y.

Rectangular Flat Plates Applied to Reinforced-Concrete Floor Slabs

SIR: In your published review of my book on reinforced concrete floor slabs in your issue for March 14, 1914, it is said that it is not clear how equation (34a) page 27, viz.:

$$A_1 jdf_s = WL/192$$

is derived from equation (34) page 26, viz.:

$$1.5\,A_1 jdf_s = WL/128$$

I think, on the contrary, that this step is perfectly clear to any one. I also think it is evident that $M_1 = A_1 jdf_s$ expresses the true bending moment of the actual cross-section A_1 of the steel and that $M_1 = 1.5\,A_1 jdf_s$ properly expresses the true bending moment of the effective cross-section 1.5 A_1 of the reinforcing steel.

I am glad to have the inconsiderable incorrectness of the numerical values in the final members of (65), page 44, pointed out. The successive numerators 0.98, 0.964 and 0.95 in (65) should be replaced by 0.89, 0.82 and 0.745 respectively. This will require the numerical factor in the denominator of equation (71), page 46, to be changed from 4.4 to 4.07, and will make equation (71) identical with (72) in case $C_2 = L_1/L_2 = 0.84$ instead of $C_2 = 0.909$, as is stated in my book.

But it is not known at what value of the ratio L_1/L_2 these equations should be identical, neither is the difference between 4.4 and 4.07 large enough to determine with any certainty which of these represents more accurately the results of tests. No such thorough analysis of the effects of the inequality of L_1 and L_2 upon test results has yet been made as to render this possible. These corrections consequently do not detract from the cogency of the conclusions reached, although they change the computed results by an amount too small to be discovered by the tests now available. The change is such as to make the computed values of the central deflection D_2 slightly larger than before. Since no general tendency of observed test deflections to exceed those computed by (71) has appeared heretofore, we may rely with considerable confidence upon equation (71) as now corrected to give us deflections at least as large as tests will show.

It is to be further noted that this equation (71) is derived for an assumed medium span of 20 ft. But larger spans than that are becoming exceedingly common, and for them the coefficient 4.07 will increase slowly with the span.

In cases where extensometer measurements have been able to be checked by comparing measurements at symmetrical points in a slab the disagreements of results which should, except for accidental reasons, be identical are so great as to cause very considerable uncertainties to attach to the accuracy and reliability of the method itself, as applied to reinforcing rods. These disagreements may be due to various circumstances—the extension of the exposed face of a rod may not be the same as that of its mean cross section; differences in rigidity of concrete may increase steel stresses in certain localities; lack of care in placing steel may have the same effect, and for other reasons.

Such circumstances render the results of extensometer measurements difficult of interpretation, as well as somewhat uncertain, and no such implicit reliance is to be placed upon them as has too often been assumed. But if the theoretic connection between stresses and deflections be accepted as established, it would appear that deflections at a few points afford a far more expeditious, economical and complete test of the strength and general behavior of a slab than any ordinary extensometer test, as well as one that is more accurate, unless the extensometer test be conducted in more detail than at present appears practicable.

Minneapolis, Minn. H. T. EDDY.

LABORATORY TESTS on a consolidation freight locomotive of the Illinois Central Railroad have recently been completed at the Locomotive Testing Laboratory at the University of Illinois. The engine was placed in the laboratory about a year ago. The tests were conducted primarily to increase the efficiency of the locomotive performance. The greatest tractive effort registered was 30,000 lb., while the highest horsepower developed was 1650. The maximum speed at which any tests were made was 45 miles per hour.